

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS

Volume XXIV

JANUARY, 1937

Number 1

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Entered as second-class matter, January 15, 1914, at the Post Office
at Boston, Mass., under Act of August 24, 1912

Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

Subscription Price, \$4.00 a Year (4 Copies)
\$1.00 a Copy

Acceptance for mailing at special rate of postage provided for in
Section 1103, Act of October 3, 1917, authorized on July 16, 1918.

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BOSTON
WRIGHT & POTTER PRINTING COMPANY
32 DERNE STREET

1127M

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ROUSES POINT BRIDGE ACROSS LAKE CHAM- PLAIN, BETWEEN ROUSES POINT, NEW YORK, AND ALBURG, VERMONT

(Presented at a meeting of the Boston Society of Civil Engineers on October 21, 1936)

GENERAL DESCRIPTION OF PROJECT

BY CARROLL A. FARWELL, MEMBER*

DESCRIPTION OF REGION

THE boundary line between the States of New York and Vermont lies in Lake Champlain for more than 100 miles, from Whitehall at the south to the Canadian boundary at the north. Within this length there is only one bridge crossing, — the existing Lake Champlain bridge between Crown Point, New York, and Chimney Point, Vermont, about 35 miles north of Whitehall. Ferries afford the only means of vehicular movement across the lake within the United States from the Lake Champlain bridge northward to the Canadian boundary. About 4 miles north of the international boundary, and more than 75 miles from the bridge at Crown Point, is the Lacolle-Noyan bridge crossing the Richelieu River, the outlet of Lake Champlain. Traffic between New York and Vermont using this bridge in Canada is subjected to the inconvenience of two crossings of the international boundary, with the resulting inspections at customs and immigration stations.

Thus for a distance of more than 75 miles there exists a water barrier to the free and expeditious movement of traffic between the

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two States. Furthermore, this water barrier affects the routing of traffic over wide areas extending long distances from the lake to the east and the west; it has been a deterrent to tourist travel between the Adirondack region of New York on the one hand, and the Green Mountains of Vermont, the White Mountain region of New Hampshire, and the lakes and woods of Maine, on the other.

The situation at the southerly end of the lake is in some ways worse than it was one hundred and fifty years ago, as, during the Burgoyne campaign of 1777, there existed a floating bridge from Ticonderoga in New York to Mount Independence, which was used by the American armies in their retreat from Ticonderoga and subsequently was employed by, and appears on the maps of, the British expedition. Until a few years ago there was available for emergency crossing the Rutland Railroad bridge near Ticonderoga, but that bridge has now been abandoned and largely removed.

A railroad bridge now exists at Rouses Point, adjacent to the site of the proposed highway bridge, which is used jointly by the Rutland and the Central Vermont railroads; however, the service offered by the railroads for local passengers is poor.

There are five ferries in the 75 miles between the existing bridge and the Canadian boundary, and four ferries southerly of the existing bridge. These ferries are of varied types, capacities and conditions. All operate on signal except the ferry at Burlington and the one about 15 miles southerly of Burlington, the usual schedule during the summer being from 7 A.M. until "dark." The ferry operation is seasonal, as during the winter ice conditions prevent navigation, and, as patronage is light during the late fall and early spring, the usual navigation season is about six months, from May 1 to November 1, although under favorable conditions some of the ferries may operate into or even through the month of November. The ferries are subject to interruption and delay, due to fog, wind and rough water, and to occasional breakdown of motive power.

During the coldest winter months a certain amount of traffic crosses the lake on the ice, particularly in the southerly portion where the crossings are relatively short. Such crossing of the lake is not possible in mild winters, and is practically never possible in the vicinity of Rouses Point, where current and wind conditions are such as to make crossing on the ice unsafe for vehicles at any time.

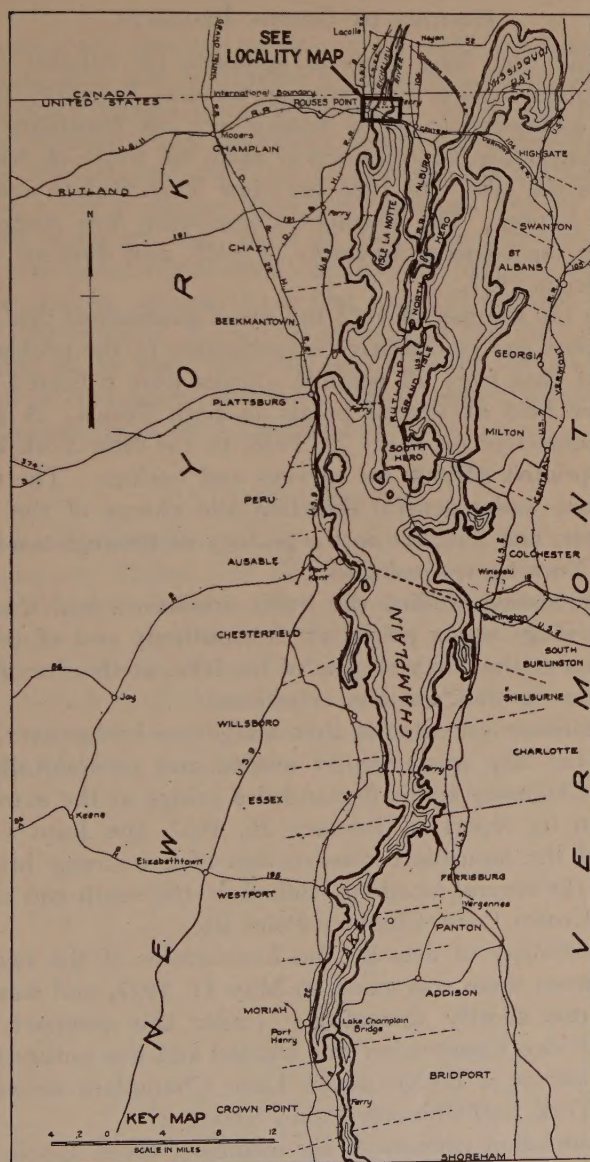


FIG. 1. — KEY MAP

HISTORY OF BRIDGE PROJECTS.

There had been long years of agitation for suitable crossings of Lake Champlain, before the Vermont General Assembly in 1923 appointed a Commission to make inquiry into the feasibility and possibility of constructing, in conjunction with the State of New York, a bridge across the lake. In 1925 the New York Senate and Assembly took similar action. The Vermont Commission was reorganized and continued by the General Assembly of 1925, and directed to bring in its final report in 1927.

Early in the summer of 1925 the two Commissions met and agreed to make a joint and co-operative investigation of the problem. Maps, statistics and data from all possible sources were collected and public hearings were held at several points in both States. A preliminary report was submitted on April 29, 1926, to the New York Legislature, which appropriated \$27,000 for surveys and borings. The engineering work was done under general direction and charge of the New York State Engineer, and a report on the geology of the region was prepared by the New York State Geologist.

The Commission studied the traffic conditions and the feasibility of bridge crossings at six points at the southerly end of the lake. A bridge location at the northerly end of the lake, at the site of the bridge now under construction, was also considered.

The Commission concluded that a highway bridge over the southerly end of the lake was urgently needed and economically justified, and also that highway traffic demanded a bridge at the extreme northerly end. In its report of January 26, 1927, the Joint Commission recommended the immediate construction of a highway bridge at the northerly of the several locations studied at the south end of the lake, namely, the Crown Point-Chimney Point site.

Under authority of acts by the Legislatures of the two States, a compact between them was made on May 11, 1927, and was consented to by Congress shortly thereafter. Under this compact, the Lake Champlain Bridge Commission was created and was authorized to construct and operate a bridge across Lake Champlain between Crown Point, New York, and Chimney Point, Vermont.

The Commission consists of six members, three appointed by the Governor of New York and three by the Governor of Vermont, serving terms of five and six years, respectively. This Commission constructed

the existing bridge and has operated it since its completion, in August, 1929.

By amendatory compact of March 30, 1935, the Lake Champlain Bridge Commission was authorized to construct, maintain and operate a bridge crossing Lake Champlain between Rouses Point, New York, and Alburg, Vermont, and this bridge is now under construction.

During the last year the State of Vermont has authorized the construction of a bridge across Missisquoi Bay, an arm of Lake Champlain, which separates the towns of Alburg and Swanton, Vermont. This bridge is being constructed and will be operated by a Vermont commission entirely distinct from the Lake Champlain Bridge Commission. This crossing of another portion of the lake will replace a ferry crossing at the same location, and, with the Rouses Point bridge, will furnish a continuous east and west highway within the United States just southerly of the Canadian boundary.

TRAFFIC STUDIES.

The Joint Commission made a study of the vehicular traffic flow in the Champlain Valley, and this study was continued by the Lake Champlain Bridge Commission, by whom the speaker's firm was employed as engineers, both for the bridge at Crown Point, now in operation, and for the Rouses Point bridge which is now under construction.

In connection with these traffic studies data were obtained which may be of interest, and information of existing crossing facilities is given in the following tabulation:

DATA OF EXISTING CROSSINGS OF LAKE CHAMPLAIN, 1935

FACILITY	Distance from Canadian Boundary (Miles)	Approximate Time of Crossing* (Minutes)	Average Toll (Aug. 10)	Estimated Annual Traffic (Vehicles)	DISTRIBUTION OF TRAFFIC		
					Canada (Per Cent)	Local, New York and Vermont (Per Cent)	Other (Per Cent)
Lacolle-Noyan bridge	4 (north)	—	\$0 60	20,000	37	24	39
Rouses Point-Alburg Ferry	1 (south)	35	1 96	7,000	4	26	70
Chazy-Isle La Motte Ferry	10	30	1 91	13,000	10	23	67
Cumberland Head-Grand Isle Ferry	24	40	1 69	25,000	9	34	57
Port Kent-Burlington Ferry	37	60	3 05	15,000	7	15	78
Essex-Charlotte Ferry	52	45	1 50	—	—	—	—
Crown Point bridge	73	—	1 00	57,000	2½	17½	80
Crown Point-Bridport Ferry	79	10	75	20,000†	—	—	—
Ticonderoga-Larrabees Point Ferry	85	15	74				
Montcalm-Orwell Ferry	87	18	73				
Wrights-Chipman Point Ferry	89	8	50				

* Including delays of entering and leaving ferries.

† Including also Essex-Charlotte.

The volume of traffic over the existing Crown Point bridge since its opening, in 1929, is a matter of public record, but the ferries and the Lacolle-Noyan bridge are private enterprises which make no return to any public agency of their traffic or revenues.

Estimates of the annual traffic for 1935 over the Lacolle-Noyan bridge and the ferries are given in the foregoing tabulation, these being based in part on short-time traffic counts and in part on other information.

As one indicator of traffic in the region, short-time traffic counts made by the Department of Public Works of the State of New York indicate that the average annual traffic for the last five years over the eight main highway routes opposite to and westerly of Lake Champlain

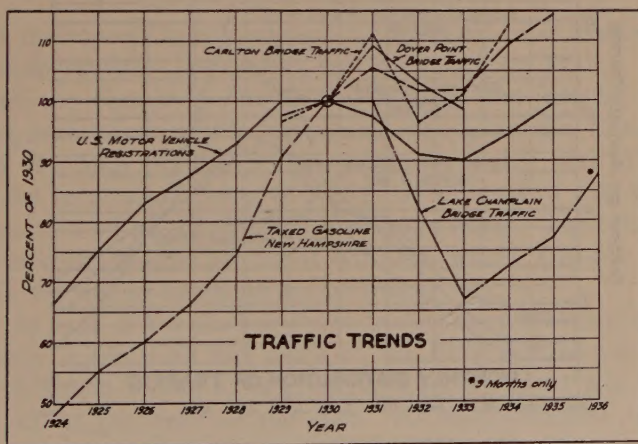


FIG. 2. — TRAFFIC TRENDS OVER PERIOD YEARS

running in a general east and west direction is in excess of 1,000,000 vehicles.

While local traffic is less important than tourist traffic, it is nevertheless a factor, particularly during the winter and spring. It is, of course, dependent on the population of the local communities. The 1930 census gives a population for the communities bordering Lake Champlain in excess of 110,000 persons.

It was concluded that the traffic at the Rouses Point bridge would be at least as much as that at the existing Crown Point bridge which carried approximately 57,000 vehicles in 1935 and has been increasing steadily since 1933. The figure for 1936 will probably exceed 64,000.

Highway traffic in the Lake Champlain region, particularly during the summer season, is largely tourist travel, and therefore it has been affected to a greater extent by the depression than would traffic in more thickly settled regions or where the vacation travel is a less important factor.

The graph of traffic trends (see Fig. 2) shows the variation in traffic at the existing Lake Champlain bridge, together with corresponding figures for other bridges, etc., all being reduced to percentages of the traffic for 1930.

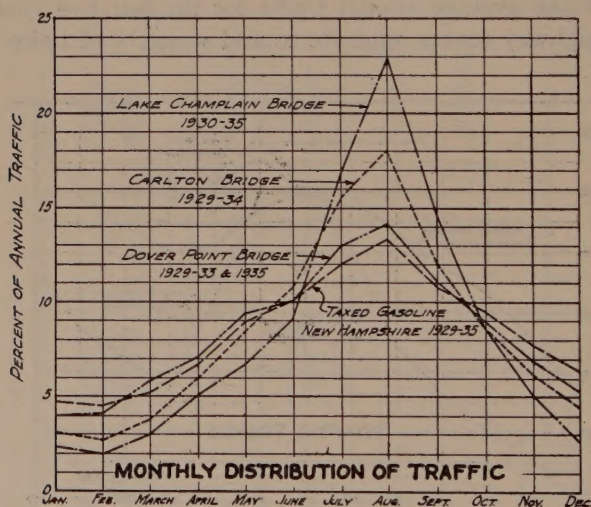


FIG. 3. — MONTHLY DISTRIBUTION OF TRAFFIC OVER CERTAIN BRIDGES

It is to be noted that the low point of the depression is indicated, generally, as in 1933, since which time there has been a steady and apparently normal growth.

In this connection it may also be of interest to observe the distribution of traffic throughout the year at the existing bridge over Lake Champlain as well as in other localities, and a graph of monthly distribution is given in Fig. 3.

It is to be noted that the summer traffic at the Lake Champlain bridge is far more important than in other regions which have a smaller proportion of tourist travel.

FINANCING

The existing bridge across Lake Champlain at Crown Point was financed partly by an advance of \$200,000 from the States of New York and Vermont, — 60 per cent New York and 40 per cent Vermont, — and partly by a bond issue of \$1,000,000. These bonds bore interest at $4\frac{1}{4}$ per cent and were to be paid serially over a period of thirty years.

The bridge has shown net revenues, after operating expenses and bond interest, during its six years of operation as follows:

1930	.	.	.	.	\$22,132	1933	.	.	.	.	\$77*
1931	.	.	.	.	19,092	1934	.	.	.	.	5,574
1932	.	.	.	.	10,128	1935	.	.	.	.	9,613

The corresponding net revenue for 1936 will be in excess of \$16,000, a figure substantially larger than for any year since 1931.

With the change in the money market, the interest rate of $4\frac{1}{4}$ per cent was unnecessarily high, and this year the Commission re-financed the loan on the bridge at Crown Point with $3\frac{1}{4}$ per cent bonds, the total issue being \$925,000, as \$75,000 of the original loan had been paid off or acquired by the Commission. This will reduce the annual interest charge by about \$10,000, which amount will be reflected in the net revenues for future years.

In the case of the Rouses Point bridge no funds were advanced by the States; application was made to the Federal Emergency Administration of Public Works for a grant, and such a grant was made in the amount of 45 per cent of the cost. At the estimated total cost of \$805,000, the grant will amount to \$362,250. A bond issue of \$500,000 was floated, bonds bearing interest at $3\frac{1}{2}$ per cent and payable over a period of twenty-five years, the first payment, however, being made in 1940. The bonds sold, in January, 1936, at 99 plus accrued interest, to yield 3.56 per cent to maturity. With estimated average gross revenues of \$60,000 per year, believed to be a very conservative figure; with operating expenses of \$11,500; and with interest charges of \$17,500 per year, there will be available for amortization of indebtedness over \$30,000 per year, a sum sufficient to pay off all indebtedness in about half the life of the bonds.

* Loss.

ARRANGEMENTS FOR CONSTRUCTION

In order to start the work within the time limit fixed by the P. W. A., a small contract for the west embankment of the bridge was advertised in December, 1935; bids were received on January 13, 1936, and the contract was awarded to Thomas F. Cunningham of Ticonderoga, New York, the low bidder, shortly thereafter. This contract covered the embankment fill, riprap and a top for the embankment, from near the west abutment of the bridge structure, a distance of about 650 feet to a point on shore where connection will be made by the New York State Highway Department. The total amount of this contract was about \$24,600.

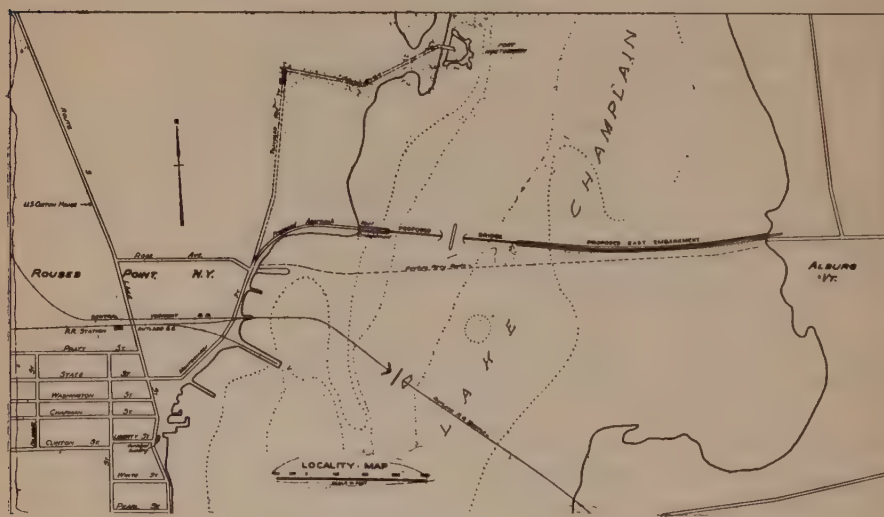


FIG. 4. — LOCALITY MAP

A contract covering the east embankment and the bridge piers was advertised in March, 1936; bids were received on April 17, 1936, and the work was awarded to Andrew Weston Company, Inc., of Long Island, New York, the low bidder, shortly thereafter. It is estimated that this contract will amount to approximately \$360,000.

Bids were received for the superstructure of the bridge on June 15, 1936, and this work was awarded to the Bethlehem Steel Company, the lowest of three bidders. The work includes the structural steel of the twelve simple spans and the swing draw span, the mechanical and

electrical equipment of the draw span, and the concrete floor for the entire length of 1,794 feet. The estimated cost of this contract is \$250,000.

At the easterly end the State of Vermont will improve the highway which leads to the ferry to provide proper access at this end. A small amount of work, namely, the surfacing and fences on the embankments, has not yet been contracted for, and arrangements will be made for this during the later stages of the work.

The total construction cost is estimated at \$672,000, the remainder of the \$805,000 being available for administration, engineering, land takings and damages, and interest during construction.

It is expected that the bridge piers will be completed shortly after the first of the year unless an unusually severe winter makes work in the lake impossible; that the east embankment will be largely completed before winter so that it can be entirely completed during the spring; that work on the superstructure will start at the site at the earliest possible time in the late winter or spring; and that the bridge should be ready for operation in the late spring or early summer. Bearing in mind that traffic from January through May probably constitutes less than 20 per cent of the year's traffic, it is not important that the bridge be operated during the early months of the year.

SUBSURFACE CONDITIONS AT THE SITE

The geology of the Lake Champlain region is interesting, and a little information from the geologist's report to the Joint Commission seems worth while.

The present site of the Adirondacks, our oldest mountains, was once occupied by the marine sea known as the Grenville Sea in which were accumulated sedimentary beds of limestones, sandstones and shales. After the formation of these sedimentary beds there came a time when rocks of igneous or molten character forced themselves up from the interior, uplifting the sedimentary limestones and shales and finally forming what may be called the Adirondack Island. Surrounding this island were marine waters, those to the east being familiarly spoken of as the Cambrian Sea, in which were deposited the lower and middle sediments of Cambrian Age. These do not occur on the New York side of Lake Champlain, but are found in Vermont. In the period following the uplift, the Adirondacks gradually sank back to some extent causing the shore line to recede toward the west, and new sediments in the upper Cambrian were laid down on the submerged

slopes of the Adirondacks. Following the deposition of these sedimentary stones there came a period when various types of limestones were formed, the last of which grades into shale. Not until after the deposition of the shales were the Green Mountains formed and elevated above sea level. With the uplift of the Green Mountains, and to some extent during later geological periods, there occurred a series of foldings of strata accompanied by considerable faulting in which the Champlain region was involved. This accounts to some degree for the very irregular shore line and steep cliffs in some sections along the lake as well as for the sudden changes in depth in the lake and various associations of different rock formations.

The great Labradorian ice sheet left abundant record of its presence in the Champlain Valley. The valley was favorably situated for ice erosion, and glacial markings show on many of the rock outcrops, but it is not believed that the present lake basin was greatly modified as a result of the glacial action. The present lake is due largely to the lower portions of the valley being clogged by glacial deposit coupled with land tilting that has taken place since the melting of the great mass of ice that once covered the valley. Directly following the glacial period the whole area about Lake Champlain was at a much lower level than at present, so that it was possible for marine waters to enter the valley from the St. Lawrence embayment, and marine fossils have been found south of Ticonderoga. The uplift of the Champlain region since glacial and marine waters occupied its basin has not been uniform. At the south end of the lake the elevation has mounted to 500 feet with an increase of about $2\frac{1}{2}$ feet per mile toward the north.

Borings taken at the site of the Rouses Point bridge indicate a hard bottom at varying depths. No core borings were taken, but the excavation at the draw pier, which has been carried to hard bottom, indicates that this is a limestone. At other piers which are under construction and which are supported on piles, the piles bring up rather abruptly at the "rock obstruction" shown on the boring records. Overlying this hard bottom is a more or less soft material through which piles may be driven or excavation made without difficulty.

The borings indicated that under the east embankment a deep layer of soft material existed, and it was expected that a considerable settlement or sinkage would result. No large sinkage occurred until the fill reached a point about three quarters of the distance from the shore to the east abutment of the bridge structure. In the remaining portion, however, the expectation of a considerable sinkage has been realized with settlement of several feet and the formation of large mud

waves on both sides of the embankment. By proceeding more slowly, it is anticipated that the fill will reach a state of equilibrium and will be floated on the soft bottom without the use of an undue amount of fill.

EMBANKMENTS

As previously mentioned the west embankment extends into the lake only a few hundred feet, for the reason that deep water in the lake, and therefore the bridge structure, is relatively close to the New York shore. On the east side, however, the water is shallow for some 3,000 feet out from shore, and in this portion of the crossing an embankment is used. The alignment was influenced to some extent by the presence of an old fill made in the lake as a part of an abandoned

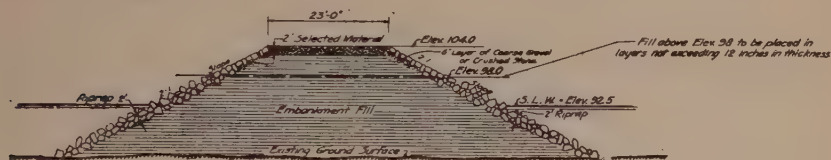


FIG. 5. — TYPICAL CROSS SECTION OF EMBANKMENT

crossing project. A top width of 23 feet and side slopes of 2 on 1 are being used.

The contractor located a borrow pit of very excellent material, including a considerable percentage of rock, which permitted the building of an excellent embankment. The riprap protection is of several kinds, as has appeared most feasible during the progress of the work. The substructure contractor purchased the old Fort Montgomery and proceeded to use it as a source of supply of concrete aggregate. It was also proposed to use the stone from the Fort for riprap with broken brick débris in the joints of the riprap and at the foot of the slope. This, however, interfered so seriously with the production of the concrete aggregate and with the use of the floating equipment on the bridge piers that riprap is now being secured from the ledge uncovered in the borrow pit on the Vermont side.

DESIGN OF THE BRIDGE STRUCTURE

BY H. J. WILLIAMS*

INTRODUCTION

THE purpose of this paper is to describe the principal features in the design of the bridge structure between the approach embankments.

This bridge structure is about 1,800 feet in length, spanning that portion of the lake crossing where the depth of water exceeds about 10 feet, the maximum depth being about 22 feet. Depth measurements are referred to standard low water in the lake (elevation 92.5). The extreme flood height of the lake is about 9 feet above standard low water. The structure consists of twelve fixed, semi-through steel truss spans, each about 122 feet long, and one through steel truss span 320 feet long. All spans rest on concrete piers.

At low water there is a waterway of about 25,000 square feet under the bridge to provide for the discharge from a watershed of about 8,000 square miles. This is adequate because of the regulation from the large storage capacity of the lake. The average velocity at the completed structure will be about one half foot per second under normal conditions, and about 2 feet per second during floods.

The bridge design is largely standard practice without many novel features, but like any large structure many details required thought and study to meet particular conditions, the most important requirement being economy. Appearance was not considered important by the Commission. After providing for the long approach embankments, only about \$265 per linear foot was available for the bridge structure.

SITE

When preliminary plans were made for a bridge at this location, the franchise was held by the company operating the ferry. The tentative location for the bridge was at the ferry crossing, which offered a convenient and fairly economical site. (See Fig. 4.) In 1935, when the plans were made for the present Commission, it appeared desirable to move the west approach northerly to take advantage of the low point of land and to make the alignment of the structure more nearly normal to

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the ship channel. At that time the Commission had not acquired title to the ferry company landing in Vermont, so that the easterly approach was to be located just north of that landing. Later the location of the approach was revised, after title had been secured, to take advantage of the filling placed by the ferry interests. The slight curve in this approach embankment is not a serious traffic hazard with the unobstructed view afforded motorists, the radius of curvature being about 2 miles.

The channel and fenders at the draw span are skewed 8 degrees from normal in order to aid navigation and to maintain a straight course from a point near the railroad drawbridge northerly past Fort Montgomery.

TYPE OF STRUCTURE AND PROFILE

Consideration was given to both a high level bridge, with sufficient underclearance to meet the requirements of the War Department, and a low level bridge with a draw span. The underclearance required for a fixed bridge would have been at least 90 feet. The low level structure was selected because its first cost is much lower. The roadway grades are better also. The length of the bridge structure was determined by two factors, — namely, the necessity of providing adequate waterway with moderate velocities, and a consideration of the relative cost of the structure and the cost and difficulty of securing a stable deep embankment. It was decided to provide 20 feet vertical clearance at the channel under the bridge at normal water level, so that most of the lake traffic will pass without opening of the draw span. Keeping the lowest part of the steel work at each end of the structure just above high water produced roadway gradients of 1.6 per cent from the draw span down to each end of the structure and for a short distance on the approach embankments, as shown on the profile.

TYPE OF DRAW

The War Department required that a clear waterway 125 feet wide be provided with adequate fenders to guide boats and tows of barges through this opening. For such conditions requiring a moderately long span with a narrow bridge and unlimited space on either side of the bridge, a swing span is best suited because of low first cost and a minimum cost of power for operation. Long fenders were required here for any type of draw, so that their cost was not an important factor in the choice.

TYPE OF FIXED SPAN

Consideration was given to many types of fixed spans varying from a trestle to fairly long spans. The trestle was eliminated because of the probable damage from ice which proves costly to the near-by railroad trestle. Estimates of cost of the maximum length of span permitted for steel stringers compared with the cost of a typical pier to support such a span indicated that the pier cost more than the span, while similar estimates for a through truss span showed the reverse to be true. This indicated that the most economical span length was between these two, or probably between 100 and 150 feet. The pony truss appeared to be the least expensive for such a span.

The cost of the various piers for the fixed spans appeared to be nearly equal, so that by making all the fixed spans alike, it was believed that a considerable saving could be made due to the duplication in fabrication and erection. The depth of the truss was determined to permit shop assembly and shipment of a whole truss to the site, thereby minimizing the cost and time of erection. This depth of 10 feet center to center of chords limited the span length to about 120 feet.

SUBSTRUCTURE

Piers

The general design of piers was based on securing an economical pier that would be safe against the thrust caused by large fields of ice driven by strong winds. Some study was given to a proposition to carry the piers on wood piles with the concrete extending only a little below low water. Such a design seemed very inadequate, as the thrust on the pier would have subjected the piles to severe bending stresses. Therefore it was decided, in general, to provide at the piers for excavating the river mud, driving timber piles to the underlying ledge or hardpan, then placing a layer of gravel at least 12 inches thick on which the concrete pier was built. At the draw pier and the easterly rest pier the ledge or hardpan was only about 10 or 12 feet below the bed of the lake. These two piers, therefore, were founded directly on hard bottom.

In designing the piers, and particularly in computing the loads on the piles or on the hardpan, consideration was given to the weight of the pier and the superstructure, the live loads on the superstructure (without impact), including the tractive force, the wind forces on both the pier and superstructure for a wind in any direction, and the probable friction on the bearings. As it is difficult to determine the force

exerted on the pier by ice, such a force was considered only indirectly, by assuming that the overturning effect would be no greater than for the other forces considered. In the early designs the pier nose was given a flat slope to relieve the ice pressure. This slope was later changed to reduce the cost. There seems to be some difference of opinion among engineers as to the size of the force, acting parallel to the bridge, to be considered at the top of the pier. The standard specifications provide for a tractive force equal to 10 per cent of the live load. This may be reasonable for some short span bridges, but the tractive force is less important in longer span bridges. For truss spans the important considerations are the components of the wind forces and the friction on the bearings.

In the abutment piers, which are buried in the embankments, consideration was given to the earth pressure acting on the back of the abutment, with only a small amount of resistance from the fill in front of the abutment. This was felt to be quite important in this location, due to the soft lake bed underlying the embankment around the abutment. The draw pier was made circular, of as small a diameter as would accommodate the track and machinery for the swing span. The round shape also is suitable for resisting the thrust caused by wind on the superstructure which may be a maximum in any direction.

Specifications provide that the design of the cofferdams and the method of placing them are left to the contractor, subject to the approval of the engineer. The bases of the piers may be placed under water, in which case the cofferdam need be designed only to be unwatered above the base with the latter acting as a seal. A tolerance of 9 inches in the position of the draw pier base and 6 inches for other piers was permitted.

The excavation is carried out under water inside the cofferdams, and the depth of excavation increased to allow for the layer of gravel backfill and for swell from the driving of the foundation piles. The specifications provide that the foundation piles be preferably of Norway pine or oak with a 13-inch butt, 8-inch tip. The piles for different piers vary in length from about 22 to 42 feet. A layer of gravel at least 12 inches thick is placed around the heads of the piles which project 18 inches into the pier base concrete.

The concrete of the pier bases is deposited under water by means of bottom dump buckets. This underwater concrete is a $1:2\frac{1}{2}:4$ mix. It was permitted to place the bases in the dry, in which case a mix of $1:3:4\frac{1}{2}$ was allowed. After the base has set, the cofferdam is pumped out and the shaft and cap of the pier placed in the dry. The shaft and cap concrete is also a $1:2\frac{1}{2}:4$ mix.

The fenders are designed to protect the swing span from damage, to guide boats and tows safely through the channel openings, and to permit boats to tie up at the bridge for customs inspection, should this procedure be desired at any time. The center fender is 35 feet 4 inches

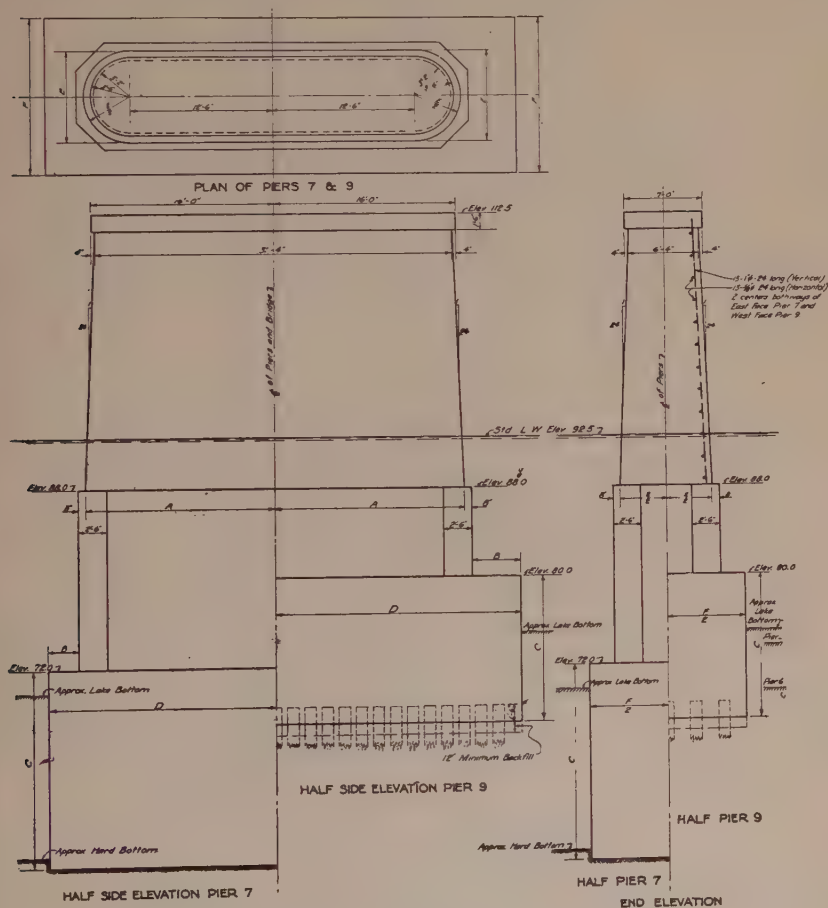


FIG. 7.—TYPICAL SIDE PIERS

wide and 320 feet 10 inches in length along each side, with the ends extended to a point. This allows about 5 feet clear on all sides of the draw span to provide for overhang or tilt of a ship and for deflection of the fender. This fender consists mainly of a series of 5 pile bents spaced 12 feet on centers, with an intermediate pile and spur shore at

each side midway between bents. There are four 12-inch by 12-inch horizontal wales along each face, surmounted by a walkway $3\frac{1}{2}$ feet wide. The fender at each rest pier consists of a straight portion 50 feet long flanked by wings 60 feet long. These side fenders have wales similar to the center fender, but supported by a single row of piles, each pile strengthened by a spur shore.

SUPERSTRUCTURE

The superstructure was designed in general in accordance with the 1935 edition of the specifications of the American Association of State Highway Officials for H-15 loading. The floor system, however, was designed for H-20 loading under unit stresses $12\frac{1}{2}$ per cent higher than those specified. Unit stresses allowed for silicon steel are those given in the 1935 American Railway Engineering Association specifications for steel railway bridges, and are one third higher than those allowed for carbon steel.

The roadway on the bridge is 20 feet wide between curbs, and the trusses are 24 feet center to center. There are no sidewalks. The expansion end of each truss is supported on a single fabricated steel rocker. The fixed span pony trusses consist primarily of 5 panels each 24 feet long, with floor beams at each of these panel points. The verticals are secondary members used to brace the chords. At the odd numbered panel points these verticals act as hangers to support the dead load of the bottom chord. At the even numbered panel points the verticals are designed to brace the top chord against lateral buckling. This matter was given considerable study to eliminate knee braces or outriggers frequently used to brace the top chords of pony trusses. The lateral force necessary to resist the buckling of the compression chord was first derived from the empirical formula, $150 (A+P)$ pounds, given by the A. A. S. H. O. specifications (where —

A = cross section area chord in inches

P = panel length in feet).

This force was later materially increased to agree with the results obtained by following the procedure suggested by Dr. Timoshenko as given in the 1930 Transactions of the American Society of Civil Engineers (Volume 94).

For most of the truss members, wide flange I-beams are used to reduce fabrication, for ease of painting and long life. The top chords, however, are composed of two channels and a cover plate, to provide

the larger radius of gyration needed. The bottom chords are filled with bituminous material to shed the water, snow and ice.

Some of the chord members of the fixed span trusses and the floor beams of all spans were designed to be made of either carbon or silicon steel. Alternate bids were taken, and the low bid showed a saving of about \$1,200 by the use of silicon steel. The price per pound bid for silicon steel was about 10 per cent, or, roughly, two thirds of a cent more than the price bid for using carbon steel exclusively. While the saving by the use of silicon steel in short span bridges, like the one under discussion, is not very great, in the case of long span bridges the saving becomes very important. The saving by increasing the allowable stresses 33 per cent at an increased price of only 10 per cent gives some indication of the opportunity for much greater savings, particularly in the future, with the further development and utilization of special steels. The recent advances along this line are emphasized in the symposium on this subject just published in the October Proceedings of the American Society of Civil Engineers.

The draw span is a center bearing through truss swing bridge of conventional design. The length of the trusses is 317 feet, center to center of end bearings, made up of 12 panels each 26 feet 5 inches long. The truss chords and diagonals are made up principally of a pair of channels turned inward. The verticals are I-beams. A double plate girder floor beam is used at the center to support the span on the center bearing. With the original design of machinery there was no space for the bottom lateral bracing in the center panels, so the chords were designed to act as portals in a horizontal plane. As the draw is not likely to be opened during severe wind storms, the design for the open position was based on a wind load of 15 pounds per square foot.

Bridge Deck

The flooring of each span is supported on four rows of steel I-beams, 6 feet on centers, framing into 30-inch silicon steel rolled floor beams. On the fixed spans the roadway slab is of reinforced concrete with a total thickness of 8 inches, including a monolithic wearing surface. The reinforcing consists of welded bar trusses and longitudinal bars shop fabricated into mats. A double step curb, 16 inches high, is composed of a concrete top and open supports, with a facing of structural steel Z-bar forming the step. The expansion joints in the roadway slab are left open, with the concrete edges protected by steel.

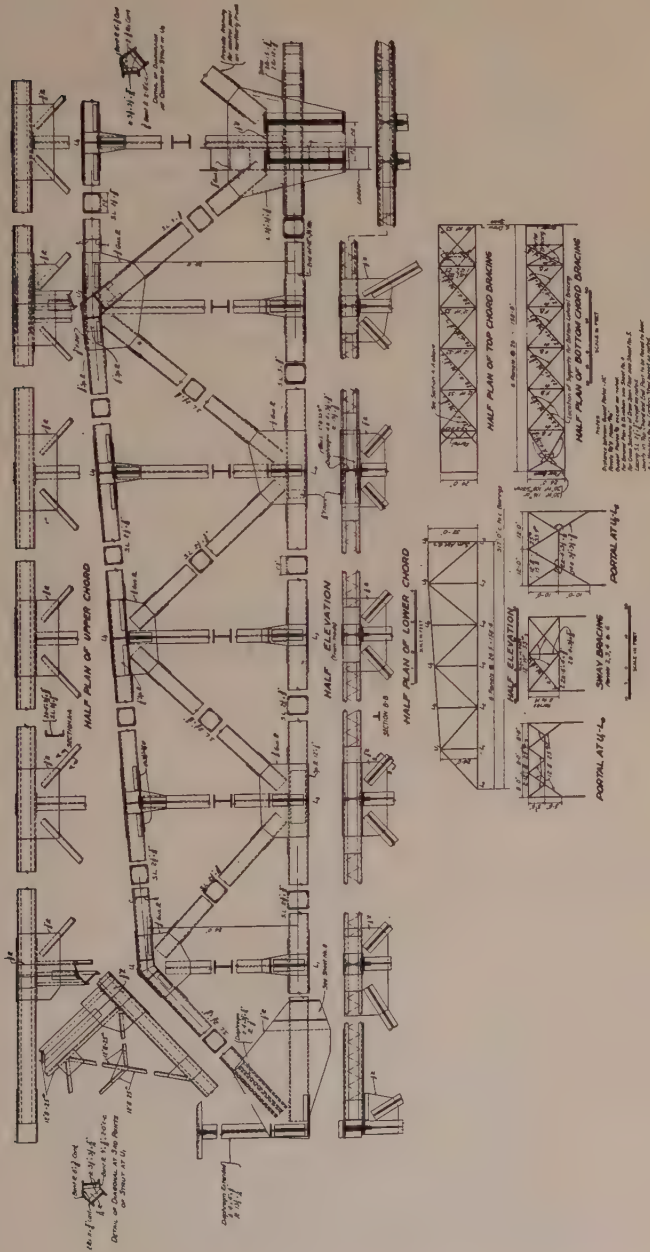


FIG. 9. — DRAW SPAN TRUSSES AND BRACING

On the draw span the flooring is a steel grid filled with concrete, weighing about 60 pounds per square foot. The steel grid is made of a series of I-beams four and one quarter inches deep connected by cross bars. This floor is a recent design with a large balanced section modulus and is adequate to span the 6 feet between stringers without intermediate supports. Its use was also permitted on the fixed spans.

There are a large number of types and designs of light weight bridge floors being developed. Many of the faults in the earlier designs are gradually being eliminated, and with the test of time the more satisfactory types are being used in many bridges. The cost per square foot of these floors is usually more than for reinforced concrete, but the saving in weight, however, may reduce the cost of the supporting structure.

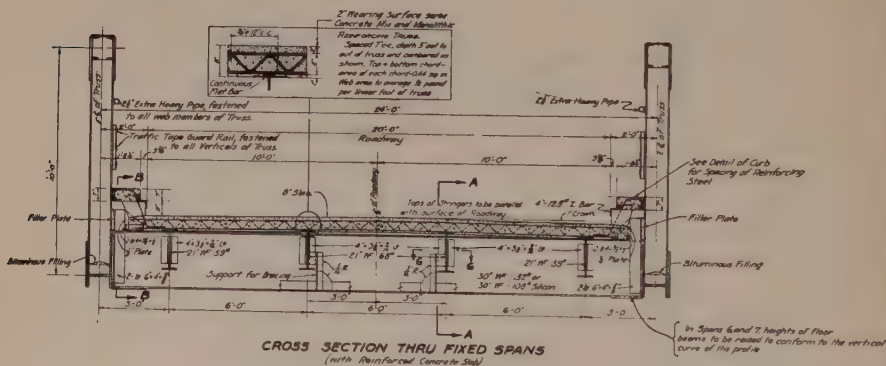


FIG. 10. — CROSS SECTION THROUGH FIXED SPANS

The railings on all spans consist of three bands of steel traffic tape guard rail surmounted by a heavy steel pipe, all fastened to the truss members.

Electrical Work

The electrical equipment includes navigation, flood, traffic and toll house lighting, power equipment for turning the swing span and operating the wedges, and interlocking equipment and control panels.

Power is furnished at 2,300 volts, 3-phase, from the west end of the bridge. Transmission is by a cable suspended under the curb of the fixed spans and by submarine cables in the lake bed at the draw span. A transformer located near the draw span reduces the voltage to 220 volts for motor operation and lighting.

The draw is rotated by a 15-horsepower, 720 r. p. m., slip-ring induction electric motor equipped with magnetic brake. The wedges are operated by a 10-horsepower, 1,200 r. p. m. squirrel cage induction motor, also equipped with magnetic brake.

The control panel for operating the draw span is located at the center of the draw span in the line of the truss. A control panel in the toll house is used to operate the lights and other equipment.

Machinery

The design of the machinery for the swing span is based upon the specifications for movable railway bridges of the American Railway Engineering Association, 1922, supplemented by the specifications for movable bridges by O. E. Hovey, 1925. The swing span is of the center bearing type with the entire dead weight carried on a center pivot, while the live load with the bridge closed is carried on wedges.

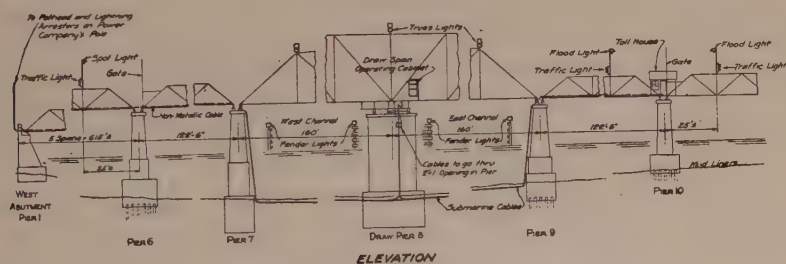


FIG. 11. — ELECTRICAL LAYOUT

The center bearing consists of two 24-inch diameter discs with spherical faces in contact in a bath of oil; one disc is of hardened steel and the other is of phosphor bronze.

While turning, the bridge is balanced against wind by six balance wheels arranged to run on a horizontal circular track 19 feet in diameter attached to the top of the pier.

The bridge-turning motor is geared to a pinion which meshes with a rack attached to the track on the bridge pier. The rack extends along one third of a circle, permitting the bridge to turn through one quadrant only. A speed reducer and a train of gears between the motor and the operating pinion reduce the speed to 2.5 r. p. m. at the pinion. The bridge can be opened in 2 minutes.

There are six wedges, two at the center and two at each end. They are operated by a system of levers, gears and speed reducers actuated

by the wedge motor. The end wedges are capable of jacking the ends of the bridge 3 inches, $1\frac{1}{2}$ inches of which would be required to overcome the drooping of the ends of the span should there be a 40-degree temperature difference between the top and bottom chords of the truss. The wedges can be withdrawn in one half a minute.

An automatic end lock is provided to align the span accurately without shock. It consists of a catch, attached to the pier, and a vertical lock bar with roller which engages the catch. The lock is so counterbalanced that if the span approaches the pier too rapidly, the roller will pass across the catch permitting the span to pass on without shock. When the span is to be opened, the lock is lifted by a lever actuated by the wedging mechanism; when the span is closed slowly the lock drops in automatically.

As the specifications permitted the contractor to substitute for the machinery shown on the contract drawings other parts of equivalent design satisfactory to the engineer, he elected to make certain changes, the most important of which is in connection with the wedging machinery. He changed the method of transmitting the operating forces from the wedge motor to the end wedges, by providing longitudinal bars to transmit the forces by tension as a substitute for the longitudinal shafting shown by the contract drawings which would transmit the forces by torque.

All bearings for shafting are provided with compression grease cups, and all moving joints are provided with a suitable means of lubrication.

If the power supply fails, the wedges may be withdrawn manually in about eight minutes, and the bridge may be opened manually in an equal length of time by four men.

Toll House

This building serves to house the toll collection facilities and as a shelter (not a residence) for those operating the draw and collecting tolls. It was necessary, therefore, to locate the toll house near the draw span. To minimize the fire hazard, the toll house is built of non-combustible material, supported on the end of bridge pier No. 10, instead of on the customary timber pile foundation. In order not to increase the length of this pier, the house is cantilevered from the pier by means of steel I-beams buried in the top of the pier concrete. This necessitated keeping the building small and light in weight. The building is made of light steel framing, with roof, walls and ceiling

composed of panels of an asbestos fiber and Portland cement compound. The floor is a steel grid filled with concrete, similar to the flooring of the draw span. All sides, floor and ceiling are insulated with rock wool. Steel doors and steel casement windows are used.

Provision has been made in the building for a desk, safe, cot, oil heater with stack, toilet with hand pump and suction to lake, and the electrical panel and lights. The fare register will be located outside of the door except during severe weather when it can be moved indoors. One roadway gate is located at the toll house, so that it may be closed at certain hours of the night to assure payment of tolls.

PRACTICAL APPLICATION OF CATALYSIS AND DISPERSION TO CEMENT AND CONCRETE

By H. L. KENNEDY*

(Presented at a meeting of the Designers Section of the Boston Society of Civil Engineers,
November 4, 1936)

INTRODUCTION

MUCH present-day concrete has certain unrecognized imperfections which, in the light of recently acquired knowledge, merit serious consideration by the concrete and design engineer. These imperfections, namely, excessive bleeding and lack of homogeneity of concrete, have marked effects on the strength, durability and permeability of concrete. It is with these imperfections in mind that the following discussion of a recent development in Portland cement technology is given. However, it must be borne in mind that this development does by no means constitute a panacea, but simply represents a definite contribution to progress in the improvement of concrete.

Many engineers are probably already familiar with some of the comparatively recent developments in industrial chemistry resulting from the use of catalytic and dispersing agents. However, few realize the importance of these developments in reducing the costs of manufacturing universally used products. Except for catalysis, innumerable organic chemicals could not be made. The point in common among all catalysts is that in every case the catalyst, without itself being consumed, acts in some mysterious way to aid a chemical reaction to run at a higher rate of speed than normally it would.

TABLE 1. — ACCELERATORS AND INHIBITORS

1. 0.1 per cent copper inhibits the corrosion of steel.
2. 0.05 per cent tetraethyl lead gives gasoline anti-knock properties.
3. 15 parts in a million of silver are used in the purification of water and alcohol.
4. 0.4 per cent manganese linoleate or a similar substance is necessary to accelerate the oxidation of linseed oil used in paint.
5. 0.001 per cent of certain organic reagents inhibits the formation of gums in gasoline.
6. Minute quantities of anti-oxidants and accelerators make possible the control of the properties of rubber.
7. Traces of plasticizers in resins and anti-oxidants in soaps and resins exert beneficial effects.

* Dewey & Almy Chemical Company, Cambridge, Mass.

Small quantities of other reagents which cannot be termed catalysts in the strict definition of the word also may have great effect in accelerating or inhibiting certain reaction. Table 1 shows typical examples of such reagents.

Turning to other fields, it is noted that the whole chemistry of the dyeing, printing, washing and absorbency of textiles and the making of inks and emulsions is being greatly changed as a result of ever-increasing new knowledge of the action of protective colloids, wetting agents and dispersing agents.

DEVELOPMENT OF TDA

With these developments in mind, and with the background resulting from the researches of the Dewey & Almy Chemical Company laboratories for the improvement of the dispersion of pigments in aqueous suspensions, it was a natural step to study what might be accomplished by aiding the dispersion of Portland cements wetted to make concretes. The story of the various steps taken to develop such an agent is too long to be told here. Suffice it to say that a great number of products were tried and over 100,000 concrete cylinders were made during an investigation, which to date has cost in the neighborhood of \$500,000. The result was TDA, — a mixture of triethanolamine salts and soluble calcium salts of modified lignin sulphonic acid, — which has today been utilized commercially for over three years.

In addition to having been thoroughly tested in various laboratories, the present TDA has been used in some 2,000,000 barrels of cement, a good percentage of which has gone into the construction of sizeable structures. Figs. 1 to 3 show typical TDA cement jobs. Today TDA is being used at the rate of 200,000 barrels per month in a total of twenty plants in the United States.

THEORY OF THE SETTING AND HARDENING OF CEMENT PASTE

Engineers are naturally thoroughly conversant with the principles and theories underlying the design of concrete mixes and concrete structures. However, due to the present-day requirements of highly specialized knowledge, few engineers have had the opportunity to acquire a working knowledge of the phenomena governing the setting and hardening of hydrated Portland cement. Since some understanding of these phenomena is necessary in accounting for the action of TDA, it may be well to describe briefly the theory.

Two theories for the setting and hardening of Portland cement have been proposed.* The first one advanced by Le Chatelier in 1882 is the crystalline theory, and ascribes the development of strength of



FIG. 1. — CONCRETE PIPE LINE



FIG. 2. — BONNEVILLE DIVERSION TUNNEL



FIG. 3. — AMERICAN RIVER BRIDGE

cement paste to the growth of interlocking crystals, as occurs in the setting of plaster of Paris. The second and most generally accepted theory was first propounded by Michaelis in 1893 and is called the

* For complete discussion see Lea and Desch, "The Chemistry of Cement and Concrete," Chapter X.

gel theory. In this theory, cement paste is considered as a supersaturated solution of hydrated compounds which eventually coagulates to form an amorphous mass called gel. This gel upon drying out shrinks and hardens, somewhat as glue does, and thus binds together the aggregates to make concrete.

Though there is little question that the silicates which constitute 70 to 80 per cent of Portland cement behave in accordance with the gel theory, the possibility exists that the aluminates upon hydration form very small crystals which are undetectable in microscopic work, due to the obscuring effect of the colloidal silicates. These compounds, however, contribute little to the strength of mortars, especially after one day, and therefore the strength-producing properties can be ascribed almost completely to the formation of gel.

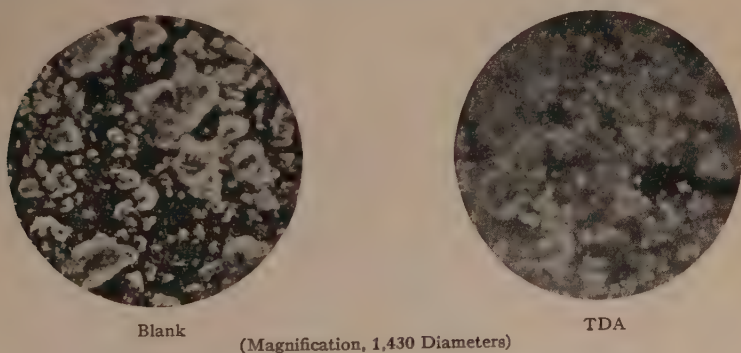


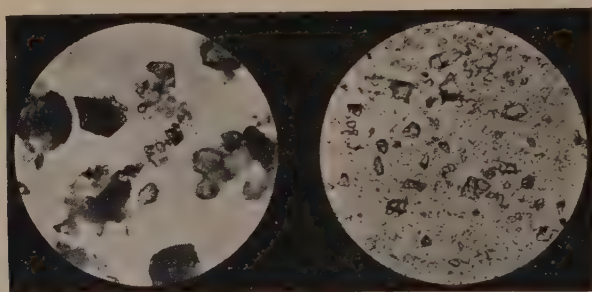
FIG. 4. — PHOTOMICROGRAPHS OF CEMENT PASTES

ACTION OF TDA

1. The triethanolamine portion of TDA acts as a catalyst of the hardening reactions. The effect of this catalyzing constituent is illustrated by the photomicrographs given in Fig. 4. The two samples of cement are the same except that the one on the right has the catalytic agent present. Both samples were allowed to hydrate for the same length of time before being photographed at a magnification of 600 diameters. Note the greater amount of gel and the finer structure of the taste in the right-hand photomicrograph. The finer structure of the cement paste is believed to be of major importance in improving the quality of concrete.

2. The lignin portion of TDA acts as a dispersing agent, as shown by Fig. 5. Note the agglomerates or clumps shown in the left-hand photomicrograph. No small particles are apparent. In striking contrast is

the right-hand picture in which we have the same cement dispersed with TDA. In this case every particle is free to act as a separate entity. In other words, more surface area of the cement is exposed to



Undispersed

Dispersed

FIG. 5. — DISPERSION OF CEMENT

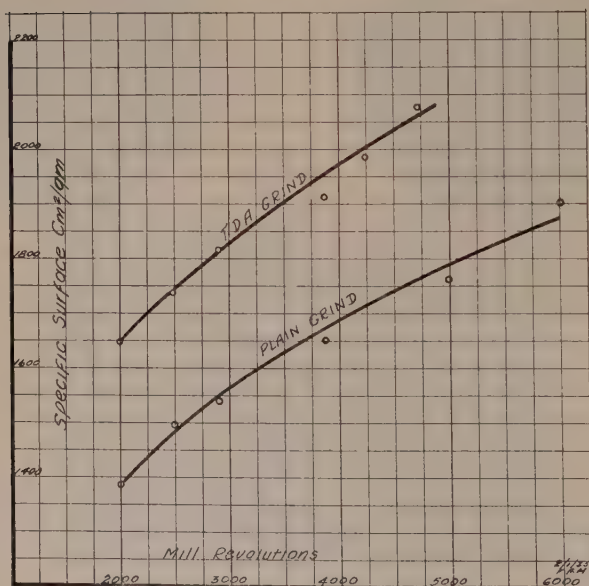


FIG. 6. — COMPARATIVE GRINDING CURVES

hydration by water, and therefore the effectiveness or efficiency of the cement is greatly increased.

3. The effect of catalysis and dispersion, as shown by these photomicrographs, on the workability and strength of concrete is obvious.

Greater effective fineness and more rapid hydration naturally produce more workable concretes, which upon hardening have higher compressive strengths. The efficiency of cement is still further increased by the fact that cements containing TDA are more finely ground. This is due to the action of TDA as a grinding aid when added to Portland cement clinker in the finish grinding mills. Fig. 6 shows a typical grinding curve obtained in an experimental ball mill.

The combined effects of catalysis, dispersion and grinding aid so increase the efficiency of cement that concrete compressive strengths are greatly increased. The average twenty-eight day compressive strength is increased appreciably, due to the TDA alone.

However, it is not the purpose of this paper to discuss in full the effect of catalysis and dispersion upon all the qualities of concrete. It is rather to discuss in detail the effects on bleeding and homogeneity, which properties have a marked but not generally recognized influence in the design of concrete structures.

TABLE 2. — COMPARISON OF BLEEDING CHARACTERISTICS OF UNTREATED AND TREATED CEMENTS

LOCATION	PER CENT BLEEDING		SURFACE AREA (CM ² ./GRAM)	
	Untreated	Treated	Untreated	Treated
South	21.8	7.3	1,460	1,560
Midwest	16.5	0.0	1,520	2,060
Midwest	15.2	4.7	1,580	1,875
East	13.3	3.5	1,955	2,120
East	13.1	2.8	1,630	1,760
East	12.0	0.0	1,620	1,770
East	11.9	5.6	1,460	1,705
East	11.2	1.7	1,620	1,780
South	10.0	2.1	1,400	1,700
East	9.6	5.1	1,755	1,840
Southwest	9.3	0.0	1,680	1,900
Southwest	8.8	0.0	1,470	1,570
Lehigh Valley	7.3	1.0	1,740	1,955
Southwest	6.7	2.1	1,775	2,050
South	6.1	1.7	1,650	1,880
Southwest	5.4	0.0	1,830	1,980
East	4.2	0.0	1,830	1,965
Midwest	0.4	0.0	1,780	1,885
Midwest				
Average	10.2	2.1	1,650	1,850

BLEEDING OF CEMENT AND CONCRETE

First of all, what is bleeding? * Fig. 7 shows what happens when thirty grams of cement and twenty cubic centimeters of water are mixed vigorously in a test tube. After a half hour or more, an appreciable volume of clear water is forced to the surface by the settling of the cement particles. The ratio of the volume of supernatant water to settled paste is known as per cent bleeding. The picture on the left shows a high bleeding, untreated cement from which considerable water has separated. The picture on the right shows the same cement with TDA interground. Due to catalysis and dispersion, a semi-rigid gel structure has been formed which prevents separation of the water. In other words, the water is used before it can escape.



FIG. 7. — THE BLEEDING TEST

Table 2 gives bleeding data for typical untreated and treated cements, the cements in each corresponding pair having been ground at the same rate. Note that the average bleeding has been reduced from 10.2 per cent to 2.1 per cent.

EFFECT OF AGGREGATE GRADATION ON BLEEDING OF CONCRETE

It must be borne in mind that bleeding of cement and bleeding of concrete are two different things. Bleeding of concrete is governed not only by the bleeding characteristics of the cement used, but also by the gradation of the aggregate, water ratio, and richness of mix used. In other words, even with a low bleeding cement, a high bleeding concrete may result if a harsh, improperly graded mix is used. On the other hand, a fairly low bleeding concrete may be obtained, even with

* Refer to "Bleeding of Cements," by L. S. Brown, Industrial and Engineering Chemistry, January, 1935.

a high bleeding cement, provided a good gradation of aggregate and a low water ratio are used. However, optimum results are obtained only when an efficient low bleeding cement is used with a properly designed mix. In all the tests made at the Dewey & Almy Chemical Company Laboratories, a uniform well-designed mix was used throughout so that results obtained indicate the effect of bleeding of cement alone.

WATER-GAIN UNDER THE AGGREGATE

Now what happens when a high bleeding cement is used to make concrete? In Fig. 8 the left-hand photomicrograph tells part of the story. Some of the water which is forced upward becomes entrapped on the under surfaces of pieces of aggregate. This phenomenon is called

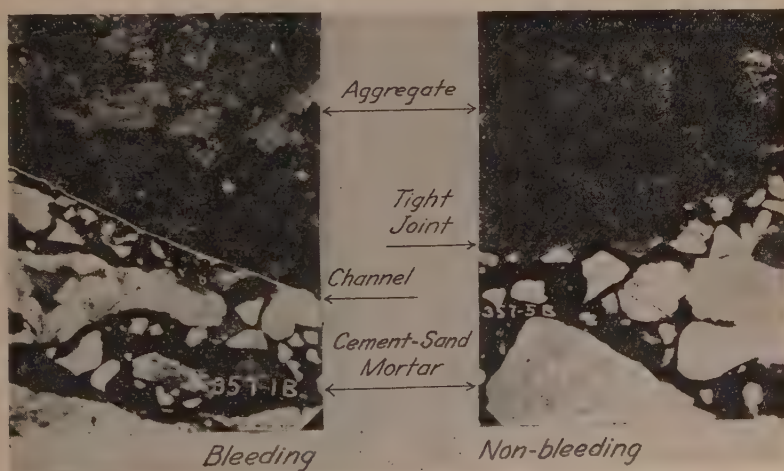


FIG. 8. — WATER-GAIN VOIDS

water-gain under the aggregate. As the reactions of hardening continue, this water is reabsorbed, leaving a capillary channel under the aggregate. When a low bleeding cement is used, no such channel or void is formed, as is evidenced in the right-hand picture.

Further evidence of the formation of these capillary channels can be found by investigating concrete cylinders tested to destruction. Cylinders which are made from high bleeding cements invariably show a clean separation of the under surfaces of the aggregate from the cement paste. Cylinders made from low bleeding cements usually show good bond between the under surfaces of the aggregate and the cement

paste as indicated by the adhesion of the cement mortar. Although no substantiating tests have yet been made, the parallel effect of bleeding on the bond between concrete and steel reinforcing rods is suggested.

PERMEABILITY

It is not difficult to visualize the effects of water-gain voids on the permeability of concrete. Where there are openings, water is going to percolate through. Furthermore, permeability and absorption are reduced by a finer structured paste.

Fig. 9 shows one of many apparatuses used to measure the permeability of concrete. The same water pressure (50 pounds per square

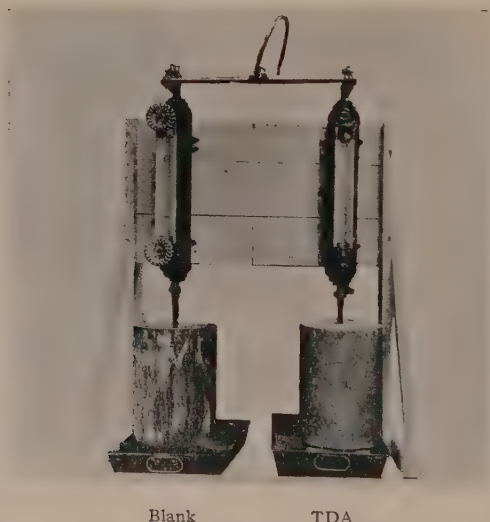


FIG. 9. — TESTING PERMEABILITY

inch) was applied to all three cylinders. The cylinder on the left was made from a normal high bleeding cement. The center one is a finely ground cement. The right cylinder was made from a finely ground cement to which TDA was added at the time of grinding. Note the water which has penetrated to the surface of the left and center cylinders.

Fig. 10 gives permeability data on untreated and treated cements obtained by this means. Striking decreases in the permeability of the treated cement concretes are indicated. These decreases are due to two factors: (1) elimination of water-gain voids, and (2) the finer

structure of the resulting concrete, both of which phenomena are attributable directly or indirectly to the catalytic and dispersing action of TDA.

DURABILITY

The durability of concrete is a function of permeability and chemical composition. Chemical composition remaining the same, it is apparent what happens when water permeates to the water-gain voids and then freezes. Hence, the best way to make concrete which is resistant to freezing and thawing is to make a fine-structured, impermeable concrete which is free from water-gain voids.

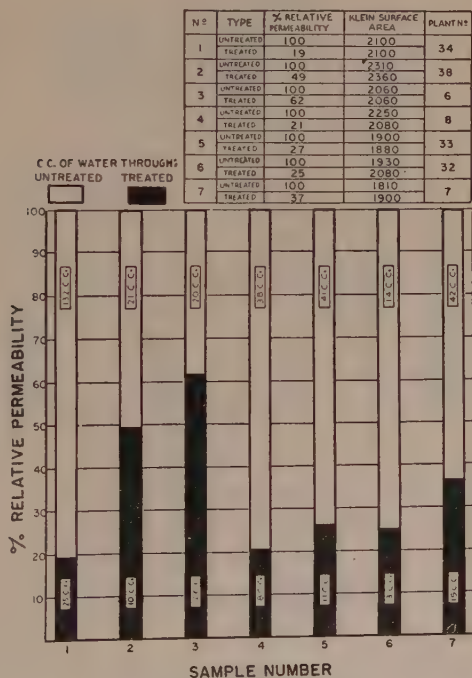


FIG. 10. — PERMEABILITY DATA FOR UNTREATED AND TREATED CEMENTS

Typical freezing and thawing data are given in Fig. 11. Cylinders tested were completely submerged in water and alternately frozen and thawed for a given number of cycles. Note that in every case the use of TDA has greatly decreased the loss in compressive strength caused by alternate cycles of freezing and thawing.

LAITANCE

As mentioned above, only part of the bleeding water is entrapped beneath the aggregate. The rest rises to the surface of the pouring, carrying with it some fines and dissolved lime. After ten or twelve hours, this surface water disappears, leaving a white scum which must be removed before pouring can be continued if a good bond is to be insured. This white scum is known as laitance.

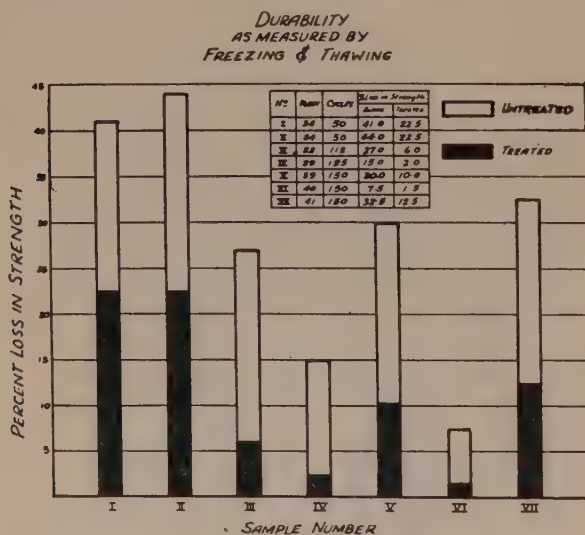


FIG. 11.— DURABILITY AS MEASURED BY FREEZING AND THAWING

HOMOGENEITY

Now what happens to the water which comes to the surface of the fresh concrete? A small amount evaporates. The rest is reabsorbed into the top layers of the concrete. This reabsorbed water has the same effect as an increased water-cement ratio and produces a weak, porous top to the pouring.

Fig. 12 shows graphically the effect of bleeding on the distribution of mixing water in the fresh and hardened concrete. It is obvious that when a high bleeding cement is used, there is a considerable variation in water-cement ratio from the top to the bottom of a pouring. As a result, strength and density vary considerably over the section. In

other words, the concrete is far from homogeneous. A low bleeding cement, on the other hand, produces concrete which is as nearly homogeneous as is practically possible to obtain.

Because of the practical importance of homogeneity of concrete, a series of laboratory tests was made to determine these variations quantitatively. Cylinders 4 by 32 inches were cast, using high bleeding and low bleeding cements. After curing for twenty-eight days, these cylinders were sawed into 8-inch lengths and broken in a compression testing machine. Fig. 13 gives data obtained from the cylinders made in this way. It is apparent that variations in water-cement ratio have

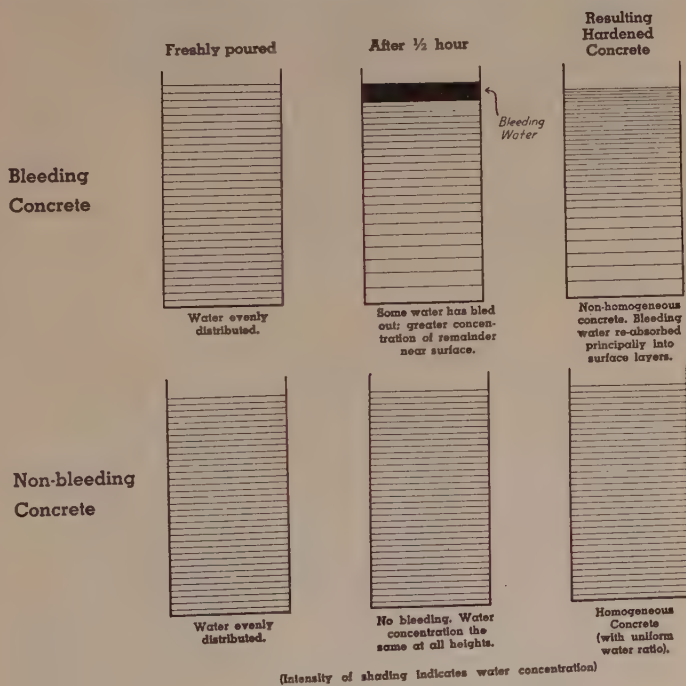


FIG. 12. — EFFECT OF BLEEDING

a considerable effect on strength, as evidenced by the fact that the top cylinders had approximately 1,000 pounds less compressive strength than the bottom cylinders.

In a paper by Prof. W. J. Krefeld, which appeared in "Civil Engineering" for December, 1933, data similar to those given above were obtained at Columbia University. In Professor Krefeld's tests, small cores were taken from various depths of an 8-inch concrete slab. Com-

pressive strengths on these cores indicate a difference of some 400 pounds per square inch between the top and bottom specimens when cured under ideal conditions. It is apparent, therefore, that even on a thin slab, such as a concrete highway, the effect of non-homogeneity is appreciable, and it is suggested that herein lies a possible answer to the many cases of surface disintegration which has taken place on our highways.

From this effect on strength, it logically follows that the concrete near the top of a pouring is less durable than that near the bottom. A striking illustration of this fact is given in Figs. 14a and 14b in which

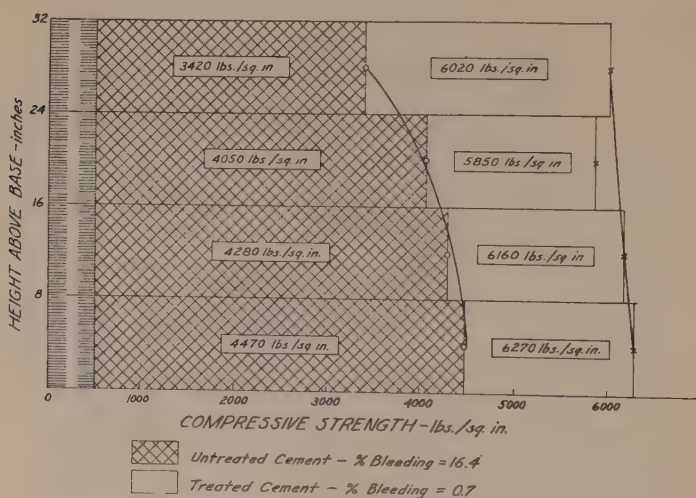


FIG. 13. — STRENGTH OF CONCRETE AT VARIOUS DEPTHS

are shown six cylinders made from relatively high bleeding cement. In every case the disintegration is worse at the top of the cylinders. The disintegration is considerably less for the specimens shown on the left, which were made from treated cements for which the per cent bleeding had been considerably reduced by the presence of the catalytic and dispersing agent. However, bleeding was not completely eliminated in the treated cements, as is indicated by the relatively slight disintegration on the top of the left cylinders. This surface disintegration will generally be found to occur in all specimens subjected to alternate cycles of freezing and thawing. Fig. 15 shows two beams that were similarly tested for durability, both being cured for ninety days and subjected to 200 cycles of freezing and thawing. The top beam



1a
TDA Cement
4 sacks/cu. yd.

1b
Control
Specimen

2c
Untreated Cement
4 sacks/cu. yd.

FIG. 14a. — CEMENT PP — 3" x 6" CONCRETE CYLINDERS AFTER
50 CYCLES OF FREEZING AND THAWING



3a
TDA Cement
4 sacks/cu. yd.

3b
Control
Specimen

3c
Untreated Cement
5 sacks/cu. yd.

FIG. 14b. — CEMENT SC — 3" x 6" CONCRETE CYLINDERS AFTER
50 CYCLES OF FREEZING AND THAWING

was made from a high bleeding cement, and is badly disintegrated. The disintegration is naturally worse at the ends and corners which are most exposed. However, the top edges of this specimen are completely rounded off, whereas the bottom edges are sharp and smooth. The bottom specimen, which has been made from a non-bleeding cement, has not disintegrated at all.

PRACTICAL SIGNIFICANCE OF NON-HOMOGENEITY

It has been established by the foregoing that the use of a more efficient cement will decrease bleeding and produce homogeneous con-



FIG. 15. — SHOWING APPEARANCE OF NO. 9 BADLY DISINTEGRATED AND NO. 10 WITH LITTLE DISINTEGRATION AFTER 200 CYCLES

cretes having finer physical structures and greater compressive strengths. Furthermore, it has been shown that such concretes are more durable and impermeable. The last two qualities, namely, durability and impermeability, are very desirable and merit practical consideration, but are not ordinarily taken into account in the actual design of a structure. The design of concrete mixes is, instead, generally based upon the strength requirements specified by the engineer, or upon the allowable unit stresses permitted by a given Building Code. What, then, is the practical significance of relatively higher compressive strengths and homogeneity of concrete?

DESIGN OF COLUMNS

First of all, in designing concrete mixes for columns where axial compressive strength is the primary consideration, attention should be given to a design based upon strength. A strength of, say, 3,000 pounds per square inch is selected as a basis, and a mix is designed to produce this strength at twenty-eight days. Unfortunately, but necessarily, the strength of a given concrete mix is determined by casting and breaking concrete cylinders which, for the most part, are not more than 12 inches high. In other words, the height is so small that the detrimental effects of bleeding are not made apparent. On the job, however, the concrete is seldom poured in lifts as small as 12 inches. In many cases where successive pourings are made within short periods of each other, the total lift may be considered as the height of the day's pouring as far as bleeding is concerned. Job conditions result in more pronounced effects from bleeding, especially where high bleeding cements are used. As a consequence, instead of the compressive strength requirement of 3,000 pounds per square inch being met in full, the bottom of the pouring will give concrete strengths of, say, 3,700 pounds per square inch, and the top of the pouring will give concrete strengths of 2,700 pounds per square inch, less than the desired strength. This despite the fact that cylinders taken on the job directly from the concrete mixer all show more than 3,000 pounds per square inch.

It may be stated, of course, from experience that concrete designed on the basis of test cylinders has in actual practice performed satisfactorily. This undoubtedly is true and can be accounted for by the fact that the allowable unit stresses as recommended by the Joint Committee Code are so conservative as to compensate for the non-uniformity of concrete. It therefore follows that the elimination of bleeding resulting from the use of a more efficient cement, with the consequent improvement in the homogeneity of concrete, will pave the way for the adoption of higher allowable unit stresses for a given ultimate strength concrete. This is obviously desirable from the point of view of construction costs.

Now, how is column design affected when it is simplified by using an arbitrary mix specified by a Building Code? In such a case, compression tests are not made, but proportions are selected which are known from long experience to give a certain minimum ultimate strength. For instance, the common 1-2-4 mix has a generally accepted ultimate strength at twenty-eight days of 2,000 to 2,200 pounds per square inch, and in design an ultimate strength of 2,000 pounds

per square inch is allowed. These figures are ultra conservative — necessarily so in view of the lack of compression tests. However, when a more efficient cement is used, a design ultimate strength of 3,000 pounds per square inch could be used for the same 1-2-4 mix. And with accurate control of the aggregate gradation and water ratio, strengths of 3,500 to 4,000 pounds per square inch could be assumed. Here, again, large possibilities for savings are apparent.

DESIGN OF FLEXURAL MEMBERS

The second important problem for discussion is that of designing reinforced concrete flexural members.

Fig. 16 shows the distribution of stress obtained by photographing with polarized light a celluloid model subject to a concentrated load at midspan. Note the concentration of stress at the point of application of the load. This concentration takes place at the top of the beam where, as a result of bleeding, the concrete is least able to carry it.

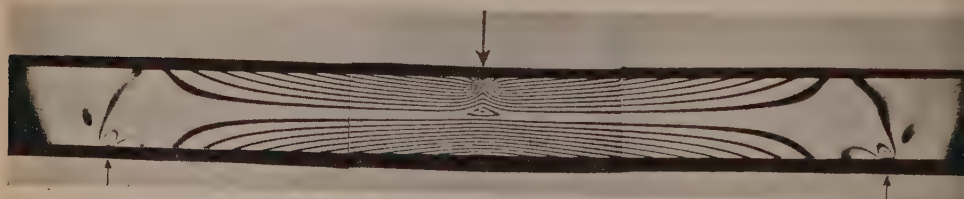


FIG. 16. — STRESS DISTRIBUTION IN BEAM

It is interesting to note that in the unloaded areas the stress lines get closer and closer together as we move from the end of the beams toward midspan. This is, of course, due to the fact that the bending moment increases to a maximum at midspan. Furthermore, at any cross section of the span the spacings of the stress lines are practically equal, — a fact which checks the theory of linear or planar distribution upon which design formulæ are based.

Fig. 17 compares linear distribution of stress, and the actual ability of the concrete to carry this stress. Note that the concrete is strongest in that region below the neutral axis where it is not called upon to do any work, and that at the top, where the actual fiber stress is greatest, the resistance of the concrete is least. Here is made apparent the possibility of increasing the allowable fiber stress by using a more efficient cement.

The problems of designing concrete structures for shear, bearing, etc., are likewise affected by the non-homogeneity and relatively low strengths of concrete. Where these factors are properly controlled, the effect of such a procedure upon the cost of concrete construction is obvious.

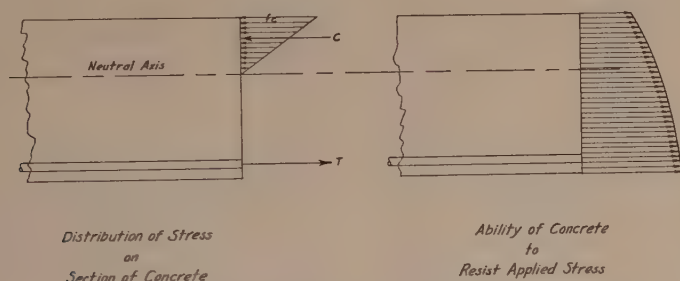


FIG. 17. — COMPARISON OF DISTRIBUTION OF STRESS IN CONCRETE AND ABILITY OF CONCRETE TO RESIST APPLIED STRESS

To sum up, as long as non-uniform concrete exists, it is doubtful if higher allowable unit stresses should be recommended. It is obvious, then, that serious thought should be given to the use of more efficient cements which will decrease bleeding and produce homogeneous concretes having finer physical structure, greater compressive strength, greater impermeability, and greater durability.

OF GENERAL INTEREST

B. S. C. E. COMMITTEE ON MEMBERSHIP AND PUBLICITY

Progress Report

During the current fiscal year of the Boston Society of Civil Engineers, 45 new members have qualified to date; and there are now on hand 15 applications awaiting action, making a total actual or immediately prospective increase in membership of 60. This is 8 per cent of the total enrollment at the close of the last annual meeting, March 18, 1936.

Bulletins describing the "Activities and Purposes" of the Society, accompanied by application blanks, have been forwarded to all members of the Society, to employers, and to heads of municipal and state engineering departments, and to all the new Associate Members of the Engineering Societies of New England, soliciting and urging the recipients to solicit new members.

Additional copies of "Activities and Purposes" and application blanks can be secured on request from headquarters, 715 Tremont Temple. The energetic co-operation of all members is urgently requested.

Respectfully submitted,

WILLIAM F. DONOVAN,
HERMAN G. DRESSER,
A. B. EDWARDS,
GEORGE A. GRAVES,
LEROY M. HERSUM,
HERBERT D. HURLEY,
KENNETH C. REYNOLDS,
FREDERIC N. WEAVER,
HARRISON P. EDDY, JR., *Chairman,*
Committee on Membership and
Publicity.

JANUARY 19, 1937.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

NOVEMBER 19, 1936. — A regular meeting of the Boston Society of Civil Engineers was held this evening at Lorimer Hall, Tremont Temple, and was called to

order by the President, Prof. H. K. Barrows, at 7 o'clock.

This meeting was the annual joint meeting with the Student Chapters of the American Society of Civil Engineers at Harvard University, Massachusetts Institute of Technology, Tufts, Rhode Island State College and Brown University, and

the Northeastern University Section of the Boston Society of Civil Engineers. About 165 members and guests attended, and 152 of them attended the buffet supper.

The President extended a welcome to the students and assured them that it was particularly gratifying to have so many attend this meeting. He expressed appreciation of the co-operation of the faculty members and the officers and committees of the student societies in obtaining so large an attendance.

The Secretary reported the election of the following new members:

Grade of Member: Wesley H. Blank, Lester M. Clark, Dexter O. Fisher, Edwin J. O'Connor,* L. T. Schofield.*

Grade of Student: Arthur R. Anderson, Boris W. Boguslavsky, Daniel W. Miles, Daniel J. Putzel, Jr.

The President then introduced the speaker of the evening, Mr. Bradford Washburn, Instructor in Geographical Explorations at Harvard University, who gave an extremely interesting talk on "Exploratory Surveys along the Alaska-Yukon Boundary." Both aerial and dog sled expeditions among mountain ranges and glacial areas, hardships in winter storms at high altitudes, and many unique and trying experiences were portrayed by motion pictures and lantern slides, interspersed with humorous anecdotes. A question period followed the talk. The applause given the speaker at the conclusion of the talk and a rising vote of thanks demonstrated the appreciation of his presentation of the account of such interesting experiences.

The meeting adjourned at 9.10 P.M.

EVERETT N. HUTCHINS, *Secretary*.

DECEMBER 16, 1936. — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers' Club and was called to order by Vice-President Karl R. Kennison. Fifty members and guests were present. Forty-two persons attended the buffet supper.

The Vice-President announced the death of the following members: Frank P. Cobb, elected a member June 15, 1904, died Sep-

tember 23, 1936; Leon O. Norwood, elected a member September 21, 1910, died June 26, 1936; James W. Woodhall, elected a member February 17, 1926, died October 5, 1936.

The Secretary reported the election of Ernest P. Demers to the Grade of Student.

The Secretary reported the recommendation of the Board of Government at its meeting held on November 19, 1936, relative to the use of the income of the Permanent Fund.

Voted, That the Board of Government be authorized to use, in so far as necessary, the current income of the Permanent Fund for current expenses. Final action on this matter will be taken at the January 27, 1937, meeting.

The Vice-President then introduced the speaker of the evening, Mr. Leslie J. Hooper, Instructor in Hydraulic Engineering at the Alden Hydraulic Laboratory of Worcester Polytechnic Institute, Worcester, Mass. Mr. Hooper was the recipient of the John R. Freeman Scholarship during the years 1934 and 1935, and has visited hydraulic laboratories and studied laboratory practices in the United States and Canada. The paper presented by Mr. Hooper was entitled "American Hydraulic Laboratory Practice." A very interesting review of the facilities and scope of the work of these laboratories was presented, accompanied by lantern slides.

A question period and discussion followed the paper. Mr. Karl R. Kennison, Prof. K. C. Reynolds, Prof. George E. Russell, Prof. J. B. Babcock, 3d, and Prof. Charles M. Allen discussed the paper.

The meeting adjourned at 9 o'clock.

EVERETT N. HUTCHINS, *Secretary*.

Sanitary Section

JUNE 13, 1936. — The annual outing of the Sanitary Section of the Boston Society of Civil Engineers was held today. The members met at Columbia Circle, Dorchester, at 1.30 P.M. The party then proceeded to the Braintree-Weymouth Metropolitan Sewage Pumping Station, where the new Diesel engines were inspected. A

* Transfer from Grade of Junior.

visit was then made to the Weymouth Water Treatment Plant, which has been in operation about six months. The treatment consists of coagulation, sedimentation and rapid sand filtration.

The Braintree Water Purification Plant was next visited. This treatment also consists of coagulation, sedimentation and rapid sand filtration.

The final inspection of the outing was at the Randolph-Holbrook Water Treatment Plant, which is now under construction.

Twenty-eight members and guests were present on the excursion. The party left the Randolph-Holbrook Plant at 5 P.M.

RALPH M. SOULE, *Clerk.*

OCTOBER 7, 1936. — A regular meeting of the Sanitary Section of the Boston Society of Civil Engineers was held this evening in the Society rooms in Tremont Temple. Preceding the meeting, the members gathered at Patten's Restaurant, Court Street, for dinner.

The meeting was called to order by Chairman Paul F. Howard at 6.45 P.M., with 23 members present. Prof. Gordon M. Fair made the motion that in the future the hour of meeting of the Sanitary Section be changed from 6.45 to 7 P.M. The motion was seconded and carried unanimously by the members present.

Mr. Elwood W. Ward presented a paper on "The Description of the New Natick Sewage Disposal Plant." Mr. Ward outlined in detail the design and construction of the plant, which consists of Imhoff tanks, trickling filters and secondary settling tanks. An interesting discussion on the construction and present operation of the plant followed.

The meeting adjourned at 8.15 P.M.

RALPH M. SOULE, *Clerk.*

DECEMBER 2, 1936. — A regular meeting of the Sanitary Section of the Boston Society of Civil Engineers was held this evening in the Society rooms, Tremont Temple. Preceding the meeting, twenty-four members assembled at Patten's Restaurant, Court Street, Boston, for dinner.

The meeting was called to order by Chairman Howard at 7 P.M. with thirty-seven members present. A paper was presented on the "Works Progress Administration River Pollution Studies" on the Blackstone, Hoosick, Housatonic and Nashua rivers. Mr. Edward Wright, sanitary engineer of the State Department of Public Health, explained the need and authorization for such studies. Mr. John H. Harding, director of the Works Progress Administration State Planning Project, explained the organization and method of carrying on the work. Mr. Ivan A. Buchanan, co-ordinator of the Works Progress Administration State Planning Project, outlined and illustrated with slides the conditions found on each of the rivers, both in regard to domestic sewage and industrial wastes. An interesting discussion followed.

The meeting adjourned at 8.30 P.M.

RALPH M. SOULE, *Clerk.*

Designers Section

NOVEMBER 4, 1936. — A meeting of the Designers Section of the Boston Society of Civil Engineers was held this evening at the Society rooms, Tremont Temple. The meeting, attended by thirty members and guests, was called to order by the Chairman, Albert Haertlein at 6.40 P.M.

The minutes of the October meeting were read and approved.

There being no further business, the Chairman presented Mr. Henry L. Kennedy, Manager of the Cement Division of Dewey & Almy Chemical Company. Mr. Kennedy spoke on the "Practical Applications of Catalysis and Dispersion to Cement and Concrete," a title which the Chairman translated as meaning "Some Characteristics of Portland Cement which have Important Effects on the Properties of Mortar and Concrete."

Despite the formidable title, Mr. Kennedy's talk proved to be very understandable, and its interest was reflected in the discussion period which followed the talk.

Mr. Kennedy illustrated his talk with lantern slides, and explained the action of certain catalysts, particularly a trade product known as "TDA," which, when

present with the cement during grinding, tends to disperse the particles and prevent bleeding. The harmful effects of bleeding near the upper surfaces of concrete surfaces was pointed out. The experimental work being done by the Dewey & Almy Chemical Company in an endeavor to eliminate this fault was explained.

The meeting adjourned at 8 P.M., with an attendance of thirty.

ANTHONY S. COOMBS, *Clerk*.

DECEMBER 9, 1936. — A meeting of the Designers Section of the Boston Society of Civil Engineers was held this evening at 110 Pierce Hall, Harvard Graduate School of Engineering. The meeting, attended by 150 persons, was called to order by the Chairman, Prof. Albert Haertlein, at 7.55 P.M.

The Chairman introduced Dr. J. P. Den Hartog, Associate Professor of Applied Mechanics at Harvard University, who presented a talk on "Dynamics Applied to Engineering Problems." This talk was an informal and very interesting exposition of various problems in vibration and the behavior of machines rotating at high speeds. Dr. Den Hartog used a great variety of models to demonstrate his talk, and, by the use of a stroboscopic light, made the vibrations which would otherwise be invisible easily seen. The model of a gyroscope, illustrating the principles used in reducing the effect of rolling in vessels at sea, was also of great interest.

A short period of questions followed, and then many of the audience took advantage of the invitation to examine the models on the demonstration bench. The meeting adjourned at 9.15 P.M., with an attendance of one hundred and fifty.

ANTHONY S. COOMBS, *Clerk*.

JANUARY 13, 1937. — A meeting of the Designers Section of the Boston Society of Civil Engineers was held at the Society Rooms on January 13, 1937, and was called to order by the Chairman, Albert Haertlein, at 6.45 P.M.

The minutes of the November and of

the December meetings were read and approved.

Upon motion, duly seconded, it was —

Voted, That the Chairman appoint a nominating committee of three members to present nominations for officers for the ensuing year. The Chairman appointed to this committee the following members: Henry Brask, chairman, Frederic N. Weaver and Lawrence G. Ropes.

There being no further business, the Chairman presented the speaker, Mr. E. L. Durkee, Assistant Engineer, Fabricated Steel Construction, Bethlehem Steel Company, Bethlehem, Pa., who spoke on the subject "Structural Steel Bearing Piles — Their Use and Capacity."

The talk was illustrated with lantern slides, and interest in the subject was shown by the large attendance of eighty-five. A discussion period followed the talk and the meeting closed at 9.30 P.M.

ANTHONY S. COOMBS, *Clerk*.

Highway Section

NOVEMBER 24, 1936. — The regular quarterly meeting of the Highway Section of the Boston Society of Civil Engineers was held this evening in the Society rooms, Tremont Temple.

The meeting was called to order at 7.10 P.M. by the Chairman, Prof. Henry B. Alvord.

The minutes of the previous meeting held May 27, 1936, were approved as read.

It was announced that the next meeting, a joint meeting with the Main Society, will be held on January 27, 1937. The speaker will be Mr. Charles M. Noble, who will speak on the "Design of Modern Express Highways from the Standpoint of Safety."

The Chairman then announced the speaker of the evening, Mr. Philip H. Kitfield, Assistant to the Project Engineer of the Massachusetts Department of Public Works, who spoke on the "Recent Highway Developments between East Boston Tunnel and Newburyport Turnpike."

The speaker displayed a large scale map of the streets and principal traffic arteries northeasterly from Boston.

The Massachusetts Department of Public Works, acting under authority from the Legislature, developed a plan to provide Boston with a gateway to the north and east. This plan involved the use of the then contemplated Sumner Tunnel to East Boston as an immediate exit from the city, then, by means of a series of highways radiating from the tunnel through East Boston, gave direct access to Newburyport Turnpike and other roads to the north after carrying traffic beyond the points of most severe congestion.

The speaker acknowledged the co-operation of other public and private engineering agencies in the conception and development of the plan finally adopted for this traffic.

During a lively discussion period which followed the paper, Mr. Channing Howard complimented the speaker and his organization on the farsightedness displayed in the development of such a system of highways. He also explained some of the studies conducted by his own organization in the same neighborhood. Many features of express highway design were brought out during this period.

After a rising vote of thanks accorded to the speaker, the meeting adjourned at 8.25 P.M. There were thirty members present.

T. C. COLEMAN, *Clerk*.

Northeastern University Section

OCTOBER 29, 1936. — A meeting of the Northeastern University Section of the Boston Society of Civil Engineers was held this evening, preceded by a dinner at Putnam's Café at which sixteen were present.

The meeting was called to order in Room 40-H by Chairman Luther Cheney, at 6.50 P.M., with an attendance of twenty-seven.

Mr. Francis H. Kingsbury, Senior Assistant Engineer of the Massachusetts Department of Public Health, discussed the work done by the Department. Control of the diseases caused by drinking water and other problems was explained. The active part taken by the Depart-

ment during the recent flood was among the interesting topics mentioned.

A general discussion by the group followed the speaker's address, after which the meeting adjourned at 8.30 P.M.

ROBERT W. SMITH, *Clerk*.

APPLICATIONS FOR MEMBERSHIP

[January 6, 1937]

THE By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ADAMS, MILTON JEWELL, Atlantic, Mass. (Age 54, b. Boston, Mass.) Experience: 1905, Massachusetts Highway Commission; 1906-07, Charles River Basin Commission; 1908-11, New York State Highway Department; 1911-14, Hawaii Loan Fund Commission; 1914 to May, 1917, New York State Highway Department; May, 1917, to June, 1919, engineer officer, United States Army; 1919-27, United States Bureau of Public Roads; 1927-32, American Rolling Mill Company; 1932 to date, New England

Metal Culvert Company. Refers to *A. B. Appleton, R. W. Coburn, G. A. Graves, E. N. Hutchins.*

BRINKLER, JOHN STANLEY, Lynn, Mass. (Age 31, b. London, England.) In 1925-26, Boston Architectural Club School, evenings; 1927-29, Building Course, Technical School, Lowell Institute, evenings; 1930 to 1934, attended M. I. T., was graduated with B.S. in architectural-engineering in 1934. Experience: 1924-29, draftsman-designer, building department, General Electric Company's River Works, West Lynn Plant; 1930, draftsman, inspection department, Associated Factory Mutual Fire Insurance Companies, Boston; 1934, designer-engineer, Daniel F. Cunningham (Parkway Development Company), Boston; 1935, with Elander, builder and contractor, Watertown, Mass.; 1936, designer-draftsman, Stadium Commission, Lynn, Mass.; at present, reinforced concrete designer. Refers to *R. S. Brown, W. H. Lawrence, Dean Peabody, Jr., C. S. Richardson.*

CRONIN, JOHN P., Medford, Mass. (Age 48, b. Worcester, Mass.) Was graduated from Worcester Classical High School, 1906, and from University of Maine, 1912, with degree of B.S. in civil engineering. Experience: From immediately after graduation from University of Maine to the present time has been with Boston & Maine Railroad as draftsman, general draftsman, assistant engineer to office engineer, which is present position. Refers to *J. B. Babcock, 3d, W. F. Cummings, F. T. Flynn, B. W. Guppy.*

DIXON, PHILIP, Roxbury, Mass. (b. Mar. 1, 1909, Vilna, Russia). East Boston High School, 1925; Lincoln Institute, class of 1932; Lowell Institute, class of 1936; now attending classes at Lowell Institute in "Advanced Structural Design" and also "Advanced Concrete Design." Experience: February, 1926-31, layout man, structural detailer and designer; 1931-36, with Groisser & Shlager Iron Works, Somerville; now in charge of structural shop and assistant to B. M. Groisser, general manager, from May, 1936, to date. Refers to *W. H. Lawrence, P. C. Malkowski, J. D. Miisch, Dean Peabody, Jr.*

ELLIOTT, JOHN L., West Newton, Mass.

(Age 47, b. Canonsburg, Pa.) In 1910 graduated in civil engineering at Carnegie Institute of Technology. Experience: 1910-11, assistant county engineer's office, Washington County, Pa.; 1911-12, inspector, Bell Telephone Company, Pittsburgh, Pa.; 1912-13, draftsman, United States Navy Department, Bureau of Construction and Repair, Newport News, Va.; 1913-17, engineer, Pennsylvania Railroad, Pittsburgh, Pa., and Indianapolis, Ind.; 1917-19, Assistant District officer, United States Shipping Board, Emergency Fleet Corporation, New Orleans, La.; 1919-22, principal assistant city engineer and track elevation engineer, city of Indianapolis, Ind.; 1922-25, city engineer, Indianapolis, Ind.; 1925-29, vice-president and sales manager, Banner Rock Products Company, Alexandria, Ind.; 1929-32, general staff manager, Industrial Department, Johns-Manville Corporation, New York, N. Y.; 1932-35, president, Elliott Roofing Company, Boston, Mass; 1935 to date, executive director, Corrugated Metal Pipe Association of New England, Boston, Mass. Refers to *G. H. Brazer, R. W. Coburn, Carl S. Ell, J. E. Lawrence.*

GRAMSTORFF, EMIL A., Lexington, Mass. (Age 44, b. Everett, Mass.) B.S. in architectural engineering, 1917, and M.S. in architectural engineering, 1936, Massachusetts Institute of Technology. Experience: 1917-20, Construction Corps, United States Navy, Grade Lieutenant (J. G.), then Lieutenant; 1920-21, contracting; 1921 to date, Northeastern University: first, instructor, then assistant professor, now associate professor of civil engineering. Refers to *H. B. Alvord, J. B. Babcock, 3d, C. O. Baird, Jr., J. S. Crandall, H. P. Eddy, Jr.*

HELD, CARL A., Marshfield, Mass. (Age 41, b. Boston, Mass.) In 1912, Y. M. C. A. Technical School (now Northeastern University); 1914, test course at General Electric Company, Lynn, Mass.; 1919, steam engineering course, Franklin Union. Experience: 1919-23, foreman and estimator for Lord Electric Company, W. S. Edmands Company, Boston; 1924, electrical engineer in charge of operation of 2,000-k.v.a. turbo plant, magnetic concentrator, and design and construction, Chateaugay, Ore., Iron Company; 1924-

25, electrical engineer, designed, installed and operated 1,500-k.v.a. hydro plant, including dam; constructed 66,000-volt transmission line sixty-eight miles long, Porcupine Davidson Gold Mines, Ltd., South Porcupine, Ontario; 1925-26, chief engineer, plant department, in charge of design, operation and construction at plant employing 18,000 persons, Western Electric Company, Kearny, N. J.; 1926-28, engineer in charge of special section of appraisal of New York Edison Company rate base case, J. L. Murrie Company, New York City, N. Y.; 1928-28, engineer in charge of rehabilitation of track, buildings, right of way, plant rolling stock of Jamaica Central Railways, Jamaica, N. Y.; 1929-31, engineer, plans, specifications and reports of industrial plants, Monks & Johnson, Boston; 1931-32, Emergency Planning and Research Bureau, Boston; 1933, state supervisor, Fire Prevention Project, under auspices of the Massachusetts Department of Agriculture; 1933 to date, supervisor, State Planning Project. Refers to *C. E. Breitbeil*, *Joseph D. Guertin*, *J. H. Harding*, *E. F. Kelley*, *W. A. Liddell*.

HUCKINS, STUART, Wellesley Hills, Mass. (Age 40, b. Duxbury, Mass.) Graduate of Harvard University, 1920; 1921-22, graduate work in sociology and economics; studies in structural design, privately pursued in recent years; since 1922 and at present, in lumber business, George McQuesten Company, East Boston, Mass. Refers to *H. S. Cleverdon*, *W. F. Pike*, *E. A. Varney*, *F. N. Weaver*.

JOHNSON, RICHARD WILLIAM, Newburyport, Mass. (Age 32, b. Newburyport, Mass.) In 1922, was graduated from Newburyport (Mass.) High School, scientific course; 1922-26, student at M. I. T.; received degree B.S., engineering administration (civil engineering option). Experience: June, August and September, 1925, assistant instructor, summer surveying course, M. I. T.; June, 1926, to date, junior engineering aid, senior engineering aid; May, 1931, rated junior civil engineer; now junior civil engineer, Massachusetts Department of Public Works. Refers to *A. B. Appleton*, *T. C. Coleman, Jr.*, *G. A. Graves*, *E. N. Hutchins*.

MAHARD, FRANCIS M., Natick, Mass.

(Age 46, b. Natick, Mass.) Was graduated from Norwich University, 1914. Experience: October, 1914, to January, 1916, instrumentman, New York Central Railroad, on Grand Central Station; draftsman, New York Civil Service Commission, on subways; assistant pilot, valuation, Pennsylvania Railroad; January, 1916, to June, 1917, designer, H. G. Balcorn, New York; June, 1917, to June, 1918, superintendent of construction, American Can Company; January, 1920, to April, 1923, designing engineer, J. D. Leland & Co., Boston, structural design of factories, schoolhouses and office buildings; April, 1923, to May, 1928, designer, Stone & Webster; May, 1928, to July, 1931, designer of power houses, Chas. T. Main, Inc.; December, 1933, to May, 1934, designing engineer on Boston Parcel Post Building, on trusses, spanning tracks, and checking all steel, Coolidge, Shepley, Bullfinch & Abbott; August, 1934, to May, 1935, designing engineer on Washington Sewage Treatment Plant, principally concrete design, Metcalf & Eddy; August, 1935, to May, 1936, Ludlow Manufacturing Company, designs and plans for mill buildings at Wilmington, Del.; May, 1936, to October, 1936, designing all types of structures for Cleverdon, Varney & Pike; at present, designer with Metcalf & Eddy, Boston. Refers to *H. S. Cleverdon*, *F. M. Gunby*, *G. C. Houser*, *W. F. Pike*, *A. L. Shaw*.

MORRISSEY, WILLIAM T., Dorchester, Mass. (Age 39, b. Charlestown, Mass.) In 1910, Bunker Hill Grammar School; graduated, 1914, Mechanic Arts High School; 1915-17, attended Franklin Union. Experience: 1920, Transitman; 1921-23, junior engineer; 1924-32, senior engineer; 1932 to present time, chief engineer, highway division, city of Boston. Refers to *H. R. Chubbuck*, *E. T. Hoisington*, *T. H. Sexton*, *S. J. Tomasello*.

PARKER, BERNARD JOHN, Boston, Mass. (Age 36, b. Cambridge, Mass.) Was graduated from Dean Academy, 1925; certificates from university extension courses about 1927-32, and technical and business courses, 1934. Experience: with E. B. Stratton, architect, for about seven months in 1926, and for about seven months, in 1926-27, with Burtman Orna-

mental Iron and Wire Works; 1927-32, for about five years, with Babcock-Davis on construction of ornamental iron, structural steel, stairs; 1932-34, with Emergency Planning and Research Bureau, Inc.; 1934-36, draftsman with C. W. A., E. R. A. and W. P. A.; for about two months to date, as draftsman with Heints Electric Company. Refers to *F. M. Gunby, J. H. Harding, R. W. Horne, A. D. Weston.*

PROTZE, HERMAN G., Roslindale, Mass. (Age 25, b. Boston, Mass.) Was graduated from M. I. T., 1932, S.B., architectural engineering; June, 1933, S.M., architectural engineering. Experience: with the Thompson & Lichtner Company, from June, 1933, to September, 1934, as apprentice, concrete engineer, then field inspection and testing, and concrete control engineer on Cape Cod Canal highway bridges, for United States Engineers; October, 1934, to May, 1935, laboratory assistant in testing materials, Massachusetts Institute of Technology, and instructor in structural materials, Lowell Institute; June, 1935, to September, 1935, consulting on structural materials; October, 1935, to September, 1936, instructor in charge of concrete laboratory and instructor in structural drawing and structural materials testing and applied mechanics, Massachusetts Institute of Technology, also instructor at Lowell Institute and Massachusetts University Extension courses; June, 1936, to September, 1936, consulting engineer and testing with the Thompson & Lichtner Company, Inc.; October, 1936, to date, engineer in charge of laboratory and testing for the Thompson & Lichtner Company, Inc., Boston. Refers to *M. N. Clair, Dean Peabody, Jr., P. S. Rice, S. E. Thompson.*

SHEA, HERMAN J., Boston, Mass. (Age, 25, b. Boston, Mass.) Was graduated from M. I. T. with degree of S.B. in civil engineering, 1933; August, 1933, to May, 1934, mathematician, United States Coast and Geodetic Survey; May to November, 1934, surveyor, on first and second order triangulation, Boston and vicinity, United States Coast and Geodetic Survey; November and December, 1934, surveyor on first order levels, Pennsylvania, United States Coast and Geodetic Survey; De-

cember, 1934, to September, 1935, junior scientific aid on adjustment of triangulation, traverse and other related geodetic operations, United States Coast and Geodetic Survey, Washington office; September, 1935, chief of party, W. N. Brown Company, Inc., running traverses for control of airplane survey of Lake Erie-Ohio River Canal site; October, 1935, to date, instructor in surveying and geodesy, M. I. T. Refers to *J. B. Babcock, 3d, C. B. Breed, J. W. Howard, J. D. Mitsch, G. E. Russell.*

WESTERGAARD, HARALD MALCOLM, Belmont, Mass. (Age 48, b. Copenhagen, Denmark.) B.S., Royal Technical College, Copenhagen, 1911; honorary Dr.Tech., 1929; Göttingen, 1913; D.Ing., Munich, 1925; Ph.D., University of Illinois, 1916; honorary Sc.D., Lehigh University, 1930. Experience: 1911-14, design of reinforced concrete, Copenhagen, Hamburg and London; 1916-19, instructor in theoretical and applied mechanics, University of Illinois; 1919-21, associate, University of Illinois; 1921-24, assistant professor, University of Illinois; 1924-27, associate professor, University of Illinois; 1927-36, professor, University of Illinois; during summers (1923, 1926, 1928, 1929 and 1932) have been with Bureau of Public Roads, United States Department of Agriculture; summers of 1931, 1932, 1934 and 1936 at University of Michigan; senior mathematician, United States Bureau of Reclamation; consulting engineer on Boulder Dam (1930-32) and for Bureau of Yards and Docks, United States Navy Department, 1935; commissioned Lieutenant Commander, C. E. C., United States Navy and Reserve, 1936. In 1922, awarded the Wason Medal, American Concrete Institute; 1935, Croes Medal, A. S. C. E. Refers to *A. Casagrande, G. M. Fair, A. Haertlein, A. B. Mason, P. C. Rulledge, H. M. Turner.*

For Transfer from Grade of Junior

CREEDON, TIMOTHY F., Arlington, Mass. (Age 30, b. Boston, Mass.) Was graduated from Northeastern Polytechnic School, Boston, in civil engineering in 1929. Experience: 1926-31, with Boston & Maine Railroad, construction on main-

tenance departments as chainman, rodman, transitman, draftsman, inspector, assistant resident engineer; 1931 to present time, as civil engineer with town of Arlington, Mass., and technical supervisor on all W. P. A. projects in Arlington. Refers to *W. J. Backes, W. F. Cummings, F. T. Flynn, W. H. Kearns.*

HAMILTON, HENRY R., Newport, R. I. (Age 29, b. Newport, R. I.) Was graduated from Northeastern University in 1932 with degree of B.S. in civil engineering. Experience: while at Northeastern University was with the engineering department of the Department of Public Works, Newport, R. I., and also for two years following graduation; from January, 1935, to present time, as Commissioner of Public Works, Newport, R. I.; has charge of Bureaus of Weights and Measures, Sanitation, Highways and Bridges, Docks and Wharves and Tree Preservation; all design, supervision and construction of public works. Refers to *H. B. Alword, C. O. Baird, C. S. Ell, A. E. Everett, Jr., A. Haertlein.*

MINICHELLO, A. ALBERT, East Boston, Mass. (Age 28, b. Boston, Mass.) 1927-31, student at Northeastern University; during co-operative periods, rodman, A. C. Peters, Inc.; transitman, New England Power Construction Company, Boston, and N. B. Snow, Barnstable; graduated in 1931, B.C.E. degree, and in 1932, B.S. degree; since 1931, estimating on bids for road contract work, the M. & O. Company, Boston; chief of party, A. G. Carey, Medford, Mass.; city of Boston, engineering draftsman, assessors' department; civil engineer, bridge and ferry division; engineer, chief of party, Planning Board; senior engineer, Works Projects Board; at present chief of party, city of Boston Planning Board. Refers to *H. B. Alword, C. O. Baird, Jr., C. S. Ell, E. N. Hutchins.*

Transfer from Grade of Student

GRADY, WILLIAM ALLEN, Winchester, Mass. (Age 23, b. Wegan, Lancashire, England.) In 1931 was graduated from Winchester High School and from Northeastern University with degree of B.S. in civil engineering in 1936. Experience: During co-operative periods, with Aspin-

wall & Lincoln as rodman, and with L. G. Brackett & Co. and George H. Dyson, Inc. as transitman and draftsman; May, 1936, to date, with United States Engineer Office, Providence, R. I., as surveyman at Hartford, Conn., sub-office. Refers to *H. B. Alword, C. O. Baird, Jr., L. A. Chase, A. E. Everett, Jr., K. F. Knowlton.*

PITTS, WILFRED C., Boston, Mass. (Age 24, b. West St. John, N. B., Canada.) In 1936 was graduated from Northeastern University with degree of B.S. in civil engineering. Experience: 1931-36, — except from April to May, 1936, when with C. T. Maney & Co., as instrumentman on South Boston Housing Project, — was rodman and draftsman with George H. Dyson, Inc.; June, 1936, to present time, with George H. Dyson, Inc., as field engineer on Bulk Oil Storage Terminal for Metropolitan Coal Company, Chelsea, Mass. Refers to *H. B. Alword, C. O. Baird, Jr., A. E. Everett, Jr., R. S. Slayter.*

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BOOK REVIEW*

"*Waterway Engineering*," by Otto Franzus. Translated by Prof. L. G. Straub. 515 pages. Published by the Technology Press, Massachusetts Institute of Technology, Cambridge, Mass.

The regulation of waterways will increasingly engage the attention of engineers, both from the viewpoint of using waterways as a means of transporting goods and from that of preventing disastrous floods. Europe, with its many congested areas, began the improvement of its rivers decades ago. Many ideas there used may be adaptable to American methods with their higher labor costs.

This English edition of a German book, by removing the language barrier, places at our disposal ideas as to German methods of waterway improvement — not that such methods should supplant American methods, but a knowledge of them should be beneficial to waterway engineers.

Engineers in this country are greatly indebted to that foremost hydraulic engineer, the late John R. Freeman, for underwriting the expense of translating the 800 pages of the German edition. The translator was the first Freeman Fellow

of the American Society of Civil Engineers, and was thus enabled not only to become well acquainted with the author of the book, Prof. Otto Franzus of the Technische Hochschule at Hanover, Germany, but to learn of German waterway practices at first hand.

By judicious choice of material, five of the eleven original chapters have been omitted in this edition, since their contents are well covered in standard engineering books in English.

"River Control" is the subject of the first part, or chapter. After providing background material, it describes types of structures and methods of construction used in regulating works. Some of these are now in use in the United States.

The second part deals with "River Mouths and their Treatment." As rivers age and additional bed-load is deposited at river mouths, the problem of maintaining a satisfactory channel through the delta increases. This may become an increasingly important problem when improving our waterways.

The third part is entitled "Effect of the Sea on the Coasts, Seashore Development, Levee Construction." Methods therein described will recall present practice in coast protection along our coasts and in levee work along some rivers and harbors.

"Weirs," the title of the fourth part, includes material on masonry dams, weirs, cylindrical and sector gates, as well as the numerous other types of gates and weirs for controlling pool levels.

The longest chapter is on "Ship Locks." The historical development is carefully traced, while the numerous types of locks are described, together with construction details. Attention is given to various types of vertical lifts.

The sixth part, on "Artificial Waterways," gives a good description of the German canal system as well as fundamental facts about canals.

This book provides an excellent exposition of European, and particularly German, methods for improving waterways, and should be a valuable reference book for waterway engineers.

* By Kenneth C. Reynolds, Associate Professor of Hydraulics, Massachusetts Institute of Technology, Cambridge, Mass.

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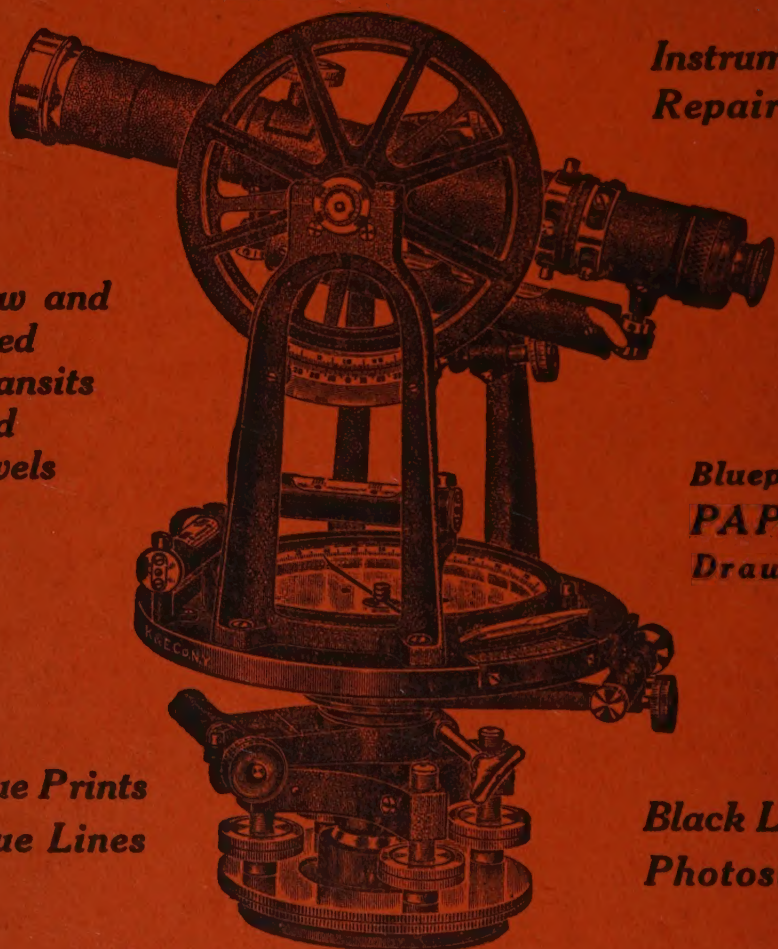
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